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Canadian
Cement and Concrete
Association

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ANNUAL REPORT
and
PROCEEDINGS
of the
Canadian Cement and Concrete Association
at the
THIRD ANNUAL MEETING AND CONVENTION
held at
Toronto, Ontario
MARCH 7th, 8th and 9th
1911

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Programme

of the

Third Annual Convention

Monday, March 6th, 1911

8.30 o'clock p.m.

FORMAL OPENING OF THE TORONTO CEMENT SHOW
St. Lawrence Arena

By the Hon. Geo. P. Graham, Honorary President of the Canadian Cement and Concrete Association.

Tuesday, March 7th

2.30 o'clock p.m.

ANNUAL ADDRESS BY THE PRESIDENT.
Peter Gillespie, Lecturer in Applied Mechanics, University of Toronto.

CONCRETE BLOCKS.
R. F. Havlick, Ideal Concrete Machinery Co., South Bend, Ind.

GRADING STONE AGGREGATE.
H. P. Bowes, Superintendent, Warren Bituminous Paving Co., Toronto

8.00 o'clock p.m.

MANUFACTURE OF PORTLAND CEMENT.
W. M. Kinney, Assistant Inspecting Engineer, Universal Portland Cement Co., Pittsburg, Pa.

THE WHITE AND THE GRAY.
Joseph M. Carrere, Blanc Stainless Cement Co., Allentown, Pa.

CEMENT SURFACES AND FINISHES.
Robt. Cathcart, Glidden Varnish Co., Cleveland, Ohio. (Paper presented by J. R. S. Scott).

Wednesday, March 8th

2.30 o'clock p.m.

CEMENT CONCRETE IN HIGHWAY CONSTRUCTION.
W. A. McLean, Provincial Engineer of Highways.

THE NECESSITY OF INSPECTION IN CONCRETE WORK.
E. A. James, Editor "Canadian Engineer," Toronto.

PREVENTION OF CORROSION IN METAL LATH.
C. W. Noble, Consulting Engineer, Toronto, Ont.

A FEW POINTS ON REINFORCED CONCRETE DESIGN.
C. S. L. Hertzberg, Engineer, Trussed Concrete Steel Co., Walkerville, Ont.

8.00 o'clock p.m.

SMOKER—St. Charles Hotel.

Thursday, March 9th

10.30 o'clock a.m.

MEETING OF COMMITTEE ON SPECIFICATIONS.

2.30 o'clock p.m.

ANNUAL MEETING OF MEMBERS.

8.00 o'clock p.m.

Joint meeting with Engineers' Club of Toronto.

ADDRESS BY REPRESENTATIVE OF THE ENGINEERS' CLUB.
J. Galbraith, Dean of the Faculty of Applied Science, University of Toronto.

BUILDING BY-LAWS AND REINFORCED CONCRETE.
Richard L. Humphrey, President National Association of Cement Users, and Director United States Structural Materials Testing Laboratories, Pittsburg, Pa.

ADAPTATION OF CONCRETE FOR LONG SPAN BRIDGES.
Frank Barber, Barber and Young, Bridge and Structural Engineers, Toronto, Ont.

PORTLAND CEMENT.
J. A. Jamieson, Consulting Engineer, Montreal, Que.

THE PRESIDENT'S ANNUAL ADDRESS.

FAILURES, AND THE LESSONS WHICH THEY TEACH.

BY PETER GILLESPIE.

It is a truism often repeated, that the public has a short memory. The lessons to be derived from great disasters are soon forgotten except by a very small number. The fatal Iroquois Theatre and Collinwood School disasters of a few years ago, so costly in child life and so much in the public mind for short intervals, showed the necessity of safeguarding the occupants of public buildings. For a time our civic authorities were active in having places of public entertainment carefully inspected. Much was heard for a season about fireproof curtains, accessible exits, fire drill and fireproof schools, but soon the public relapsed into its old ruts. Inspection became less searching and the authorities more forgetful, and the public continues to run daily the same risks that the victims of these disasters ran. It sends its children to the same schools and attends itself the same places of public worship or entertainment and only becomes conscious as to the chances it is taking when some other horrible calamity occurs. Truly the public has a short memory.

It is chiefly because humans are humans, and not machines, that we continue to take chances. A machine can be constructed to do a given thing in a specified way an endless number of times. Whether its work be punching or drilling or cutting, it does it with machine-like precision. It never tires, never grows careless and is never actuated with a spasm of over-zeal or indolence. It is never tempted to scamp its work. It is never influenced by the desire to make excessive profits, and never knows the flattery of the multitude or the sting of adverse criticism. It never forgets, never flatters, never tempts, never cajoles, never bluffs and never pleads. But men are differently constituted. They possess the human traits. They are influenced by example, possess passions and emotions, cherish hatred, remember injuries and forget the lessons which great crises in their experience ought to impress on them. The attention of engineers, architects, builders and building departments has been called to the lessons which are taught by the failures of structures designed and erected by them or under their supervision. It is not that the field is a new one that this paper is devoted to so hackneyed a topic, but to emphasize once again, first those elements which have contributed to failures, and secondly those corrective or precautionary measures which will tend to prevent their re-occurrence.

Especially in the use of reinforced concrete has the general reader's attention during the past five years been called to a rather large list of failures, all of them more or less serious, and not a few of them having fatal results. Reinforced concrete has been comparatively new in the building art and in its fabrication is very different from other materials with which the constructor is familiar. Unlike steel or wood, its strength increases with age. It is poured into forms, at which time it is plastic, and has to be sustained until it acquires sufficient strength to support itself. It consists of two materials, not one, and since the disposition of these, with respect to each other, vitally affects the strength of the product, exercise for great care in this placing is necessary. More, perhaps, than of any other building material is this true—it possesses great capacity for injury in the making and placing. Following are cited a number of failures of reinforced concrete structures which during the past twelve months or so have occurred, and reference to which in the engineering press has come to the writer's notice. They represent typical cases, and an examination of the list will enable us to classify the causes under a few general heads.

The comparatively recent announcement that the great dam across the Colorado River at Austin, Texas, is to be rebuilt, has served to recall its failure over a decade ago. It will be remembered that that failure was announced to the world at first as a serious reflection upon the engineering profession because some hydraulic engineers of eminence had been connected with the work. Subsequently, however, when the whole history had been investigated, it was found that the authority of the engineers had been interfered with to such an extent by the city officials in control, that their responsibility had been practically nullified. Indeed, one prominent engineer had resigned rather than have his name further connected with a work over which he had no control.

On February 28th, 1910, a reinforced concrete arch of three spans over the Flat Rock River at Edinburgh, Ind., collapsed during an unusually heavy flood. The design had been furnished by a well-known bridge company, but the Bridge Commissioners, in their desire to economize, and with a wisdom commensurate with their experience in such matters, decided to omit the piling underneath the abutments and to carry them instead to a somewhat greater depth. And so the bridge was built. The materials supplied and the workmanship throughout seemed to be excellent, but in the season of heavy flood the piers were undermined by scouring and the structure collapsed. There does not appear to be any other cause of failure than the insufficiently supported piers, and the responsibility of course must rest on those who ordered the modification of the original plan.

A concrete dam at Fertile, Minn., was washed out by floods early in April, 1910. The trouble is attributed to the fact that the foundations were not laid sufficiently deep to prevent scouring and undermining, and failure ultimately ensued.

The partial failure of the Bayless reinforced concrete dam at Austin, Pa., in January, 1910, exemplifies a trouble of a slightly different character. The dam was completed in December, 1909, at a cost of upwards of $70,000. On completion it was observed that there was one small crack 1/16 inch wide, extending from the top of the dam to the ground level. Consequently others similar in appearance developed, and during a heavy freshet on January 23rd, 1910, a section of the dam between vertical cracks, under the thrust of the impinging water slid forward some 18 inches. This movement covered a period of some eight hours and then stopped. Investigation disclosed the information that the failure was due primarily to the fact that the dam was founded upon a bed rock, the successive strata of which were separated by layers of shale or clay. The impounded water, having worked itself under the foundations, had softened the clay, with the result that the upper stratum carrying a portion of the dam had moved forward on the lower. It was stated that the weakness of the concrete was doubtless due to the fact that much of it had been hurriedly placed, part of it in freezing weather. The anchor bolts, which had been grouted into the foundation rock, had moved outward with the rock into which they were anchored.

On April 7th, 1910, the collapse of a concrete roof under construction at the new car barn of the Shore Electric Road at Saybrook, Conn., took place, and resulted in the death of one man and the injury of two others. The roof was a four-inch slab of reinforced concrete on girders, about eight feet on centres and of 37 feet span. The last work had been completed about ten days and the forms were being removed. It was believed that the premature removal of the forms and the excessive loading of the green roof slab with roofing material were the joint causes of the accident.

On July 13th, 1910, one of the columns of the concrete groin arch roof to the filter chambers in course of construction at Owen Sound, Ont., collapsed while the centering beneath was being removed. Two men were quite seriously injured. The accident apparently was due primarily to the early removal of forms, combined with the fact that the 18 x 18 inch columns on two sides of the square roof of the arch were quite incapable in themselves of resisting the arch thrust. The forms were removed in only four days after pouring, notwithstanding the fact that seven days was the minimum specified time for removal.

The upper part of a reinforced concrete chimney under construction at the plant of the American Woollen Co., South Royalston, Mass., collapsed on April 9th, 1910, causing the death of two men. The chimney was to have been 105 feet high, with an inside diameter of 4 feet and an outside diameter of 4 feet 8 inches at the bottom, and with walls varying in thickness from 8 inches at the bottom to 4 inches at the top. The forms used in construction were in two sections, each 3½ feet deep. The procedure was to fill the upper form, then to loosen the lower and set it above the upper for filling. This took one day. Next morning the form in the lower section was loosened and it was placed on top and filled. Thus the concrete in any 3½ foot vertical section had less than 24 hours in which to set before its side-supporting forms were removed. The accident occurred when a height of about 70 feet had been reached, the section last uncovered, then only 20 hours old, caving in and carrying the workmen to the ground with it. It is reported that the temperature the day before the break had taken a sudden drop, but it was not at any time below freezing. The failure was undoubtedly due to loading a very green concrete before it had acquired sufficient resisting power, as the materials were good and the design and execution satisfactory.

A reinforced concrete grain elevator of typical design failed under a normal pressure of grain at Springfield, Ohio, on October 24, 1910. As has been rather common in elevator construction, where a battery of cylindrical units has been constructed, the unused spaces, external to the cells, but lying within the tangent walls, had been utilized as auxiliary bins. No one saw the beginning of the collapse, so that its exact behaviour cannot be stated but from the appearance afterwards it was evident that the lower section of the wall, where the pressure was greatest, was forced out under the pressure of the wheat. This portion sheared off clean at the line where the straight wall connected with the circular wall of the larger bin. The weak point in the structure was that the horizontal rods in the straight wall were not connected to those in the circular, nor were they tied back for any distance into the concrete of the circular walls. It was stated that the plans showed the rods in the straight wall securely fastened into the circular bins and that the failure to so fasten them was due to the negligence of the foreman.

On November 22nd, 1910, a four-storey reinforced concrete building erected for the Henke Furniture Co., Cleveland, Ohio, suddenly collapsed, throwing one of the walls over a two-storey frame building next door and so crushing the structure as to cause the death of four of its occupants, and the serious injury of seven others. It was of reinforced concrete column and girder construction, with hollow tile and concrete floor system, and brick curtain walls varying in thckness from 21 inches at the basement to 13 inches at top floor. A commission of enquiry was immediately appointed, on which were representatives of the Builders' Exchange, the Cleveland Engineering Society and the Cleveland Chapter of American Institute of Architects. This commission was empowered to take evidence and to consult every available source of information that might explain the cause of failure. After the wreck had been carefully examined, the design was checked over, and the witness examined, the finding was announced. The failure, it stated, was due primarily to the premature removal of forms in the third storey. It fixed the responsibility on the architects, the contractor, the owner and the Department of Buildings for the City of Cleveland. The architects were adjudged responsible in that they did not give sufficient attention to the materials, and in that they did not give adequate supervision to the work of construction. The contractor was adjudged culpable in that he employed foremen who were entirely ignorant of the intent of plans and specifications or of the nature of the materials they were handling. In consequence the sand was inferior, the forms were removed prematurely, and that, regardless of weather conditions, the green concrete was regularly overtressed, the members were not of the size called for in the plans, the concrete composing them was open and porous, and sawdust and shavings were found in the bases of columns. In addition, it developed that less cement had been delivered to the building than would have been necessary to construct it had it been built as planned.

The owner was deemed responsible in that he had not employed a special concrete inspector on the work, as required by the building regulation.

The Department of Buildings was held responsible in that it had ignored the requirements of that portion of the building code which makes it necessary that the owner employ a special reinforced concrete inspector. It thus appears that the Building Department, rather than the building code, was at fault.

I have chosen to classify the causes of failure in the above cited cases as follows:

1. Interference with a suitable design by those in authority, but not possessing a knowledge of engineering practice.

2. Defective design.

3. Inferior materials.

4. Ignorant or wilful disregard of specifications and plans.

The first of these is unfortunately of too common occurrence. In all human probability the Austin dam and the Edinburgh arch would be standing to-day, but for the intervention of the "cock-sure aggressive" individual who, when clothed with a little brief authority, becomes a paragon of wisdom on everything under high heaven. This type is found frequently in our municipal councils, and it is to be regretted that engineers of wide experience and good judgment sometimes permit themselves to be dominated by them. An engineer's judgment may be in error, but it is not more likely to be productive of good results if it be corrected through consultation with other engineers of equally good standing, than if it be reversed by men entirely untrained in the problems of design and construction.

Fortunately, the day of unsafe design in reinforced concrete is becoming a thing of the past. The design of reinforced concrete, like the design in wood or steel, has been reduced or is being reduced, to a standard basis on the proportion and strength of materials which constitute it. There is, therefore, no reason to-day why the average practising engineer should not acquaint himself with concrete designing, so that at least he can finish his plans with some such detail as he does those for his structures in steel. He may, if he prefers, leave the details to his contractor, who, like the contractor for steel, can make them according to the standards of his practice. He must exert every care in informing himself as to the character of the materials he must employ and of the foundations upon which he proposes to erect his structure. The dams at Fertile, Minn., and at Austin, Pa., furnish illustrations where disaster might have been averted by a careful examination of the underlying strata prior to construction. The designer must recognize, too, that the safety of his design depends upon the constructor as well as upon himself, since concrete construction is not fixed as is that in steel. An editorial in the "Engineering News," speaking of this phase of a designer's responsibility, asserts that the "design cannot be sent from the drawing table with perfect confidence in its precise reproduction in the structure. It is the joint product of the man in the office and the man in the field, and any design which fails to recognize this is a poor one, no matter how nearly it may conform to accepted standards. No engineer who is not prepared to supervise the construction of a reinforced concrete structure is justified in designing that structure as closely to the safety limit as he is when the construction is to be under his eye."

The prevention of the utilization of inferior material is the work of the engineer, the architect and the inspector. No reinforced concrete work of any magnitude should be constructed without a capable, conscientious inspector. The Hencke building suffered from inferior material, as it did from almost every other malady to which reinforced concrete is heir. The remedy for this is vigilant, constant inspection. Failure resulting from the premature removal of forms could invariably have been prevented by the exercise of intelligence and a little precaution. The Hencke building disaster, the Owen Sound filtration plant accident, the car-barn roof failure at Saybrook, Conn., and the chimney collapse at South Royalston, Mass., were all preventible if careful examination had been made before stressing the concrete, which, possibly due to temperature conditions, had not yet acquired its preliminary strength. The failure of the grain elevator at Springfield, Ohio, was due to the ignorance of a foreman. Any foreman who appreciated the properties of the materials he was handling would not make the blunder that was made.

The remedy for these evils is the employment of the experienced, intelligent, painstaking inspector. He sees that the forms are cleaned before the concrete is poured, that the ingredients are correctly proportioned, that the steel is properly placed and in correct amount, that the members are of dimensions called for by the plans, and that the forms are not removed until the material has acquired sufficient strength to be self-sustaining. The capable inspector is essential to safe construction.

[*This Association is not responsible as a body for the opinions expressed in its Papers by Authors.*]

CONCRETE BUILDING BLOCKS.

BY ROBERT F. HAVLIK.

In speaking of concrete blocks, I am touching upon a subject which has been hatched and rehatched many times over, and yet in spite of all that has been said on this subject, we are learning so much more each year in the way of new methods of making concrete products, that we wonder how we could begin to have been satisfied with the old methods which in the light of the present day appear so crude.

In discussing this subject I shall endeavor to confine myself to the essentials only, and not burden you with minor details which you are familiar with.

I well remember the crude-looking blocks exhibited at the Chicago Cement Show in December of 1906. Water, when poured on same was absorbed instantly. To-day as we inspect the products exhibited at our various shows we see an entirely different material—one that is dense, hard and waterproof. Five years ago the so-called concrete blocks were used in cheap buildings only, in foundations and underground. Only cast stone was considered for good work where a cut stone effect was desired. A gradual change has taken place since then, however, and the molded concrete block is now being used in some of the finest buildings. Cast stone is no longer as popular as it was then, not because it is not as good as the molded concrete block, but because the block can now be made as good as cast stone at a much lower cost, and since cost is always a prime consideration when the quality is the same, the molded concrete block is destined to play an important part in the building operations of the future. The best testimony we have of this is in the magnificent buildings of the Barber Estate at Barberton, Ohio. These are being built of molded concrete blocks, made in standard machines and special wood and gelatin molds. When such men as Mr. Barber, who want a beautiful effect regardless of price, select molded concrete blocks in place of all other materials, there must be a great value in properly made concrete blocks.

I am well aware of the fact that an immense amount of inferior work has been done in concrete blocks, but mistakes have been made in every new industry, and it is but natural that mistakes were made in the concrete block industry. But we have passed the mere experimental stage of this industry and concrete blocks are now a staple building material, and if we continue to improve them in the future as we have done in the last few years, they will soon be in greater demand than any other one building material. Concrete blocks have everything in their favor. They make a house that is cool in summer and warm in winter, their strength increases with age, whereas nearly all other building materials deteriorate with age; they can be made waterproof, and, last but not least, homes built of blocks cost less in the last analysis than frame buildings, because they require no repairs. But for this fact many a building now being put up of concrete blocks would be built of frame on account of the prohibitive cost of brick and stone. Thus the concrete block is improving the building construction of to-day both in beauty and permanency.

The concrete block is here to stay. The severe criticism to which it has been subjected in the past has resulted in the greatest possible good, in that it has forced the block manufacturer to improve his product, and if we continue improving it as much in the future as in the past, we will soon place it beyond the criticism of its present severest opponent.

We can improve this industry best by paying more attention to the selection of the materials used and the methods employed in the manufacture of the blocks and placing on the market the best product we can make. The cement should be a Portland cement that will pass standard specifications. So much has been said about the proper selection of aggregates for concrete work that I will not burden you with needless repetition any more than necessary. The most common aggregates used in concrete blocks are sand and gravel, although crushed stone is sometimes used in place of gravel and stone screenings in place of the former. There is considerable objection to both of these, however, especially the latter, on account of the crusher dust that is always present and clings to the stone particles and prevents the cement from properly bonding with same. It has been my experience that gravel and sand produce a denser and better looking concrete. In many localities, however, sand and gravel cannot be had, except at prohibiive prices, whereas crushed stone may be very plentiful. In such cases there is no recourse except to use this material. Whatever the materials, the finer, which is usually sand, should be well graded from 1/4 inch to 1/50 inch or 1/100 inch. The coarse material should be well graded from 1/4 inch to 3/4 inch, not to exceed 1 inch. The largest aggregate should never be larger than one-half the thickness of the thinnest wall of the block. Where a pit run of sand and gravel is used the material should range in size from 1/100 inch to 3/4 inch. Both fine and coarse aggregate should be free from clay, dirt or fine dust, as these but tend to decrease the strength of the block. The water should be free from alkalies.

The next important consideration is that of proportions. There are three principal methods in vogue for determining the proper proportions—the "void" method, proportioning by trial mixtures, and proportioning by a study of the mechanical analyses of the various aggregates. The first method is universally acknowledged to be little better than a guess, and has been discussed so frequently that I will pass it by without further comment.

For any given materials which are to be used in their natural state the second method of proportioning by trial mixtures will be found very satisfactory, but will not show what other changes in the sizes of the aggregate could be made. The procedure in this method is very simple. First get a good scale and a rigid cylinder, say a piece of 8 inch or 10 inch pipe, 12 inches long or so. Weigh out and mix together carefully any arbitrary amounts of the cement, sand and gravel to be used, and make the consistency the same as that of the concrete to be used in the blocks. Place this mixture in the pipe, tamping same very carefully in thin layers. When it is in place note the height of same in the pipe. Then weigh out the same amount of cement as before, and the same total weight of sand and gravel, but vary the relative amounts, and repeat the operation, using the same consistency as before. Note the height this mixture occupies in the pipe. That mixture which takes up the least space in the pipe is the best for those materials as they are.

The third method, while a little more difficult for the beginner, when once understood proves to be the simplest of the three. It permits of the immediate determination of the best combinations of raw materials in their natural state and also how they can be improved upon by adding or screening our certain size aggregates. In 1910 Mr. Wm. B. Fuller made an extensive series of experiments on the comparative strengths of different proportions of concrete aggregates. All the aggregates used were screened through various size screens after having been carefully mixed, and the percentages by weight of the materials passing screens of certain meshes were carefully recorded and plotted to curves with the distances on the vertical ordinates representing the percentage by weight passed through screens whose mesh is represented by distances along the horizontal lines or ordinates. He found that for any given amount of cement the strongest concrete was produced from that combination of aggregate whose mechanical analysis plotted to a curve as above, formed a parabola, passing through the zero ordinates and the intersection of the diameter representing the largest stone with the 100 per cent. ordinate. This is discussed in detail in a special chapter by Mr. Fuller, in Thompson &

Taylor's treatise on "Concrete, Plain and Reinforced." The percentage by weight of the aggregate smaller than a given size can be easily calculated by the formula for the parabola:

$$d = \frac{P^2D}{10000} \text{ or } P = 100\sqrt{\frac{d}{D}}$$ in which P = per cent. of mixture smaller than any given diameter.

d = any given diameter.
D = largest diameter of stone.

From this it will be seen that the per cent. of aggregate smaller than a given diameter, say ½ inch, and larger than a second diameter, say ¼ inch, is obtained by subtracting the percentage smaller than ¼ inch from the percentage smaller than ½ inch. The following table is figured out on this basis and shows the percentage of the various sizes for any given mixture up to 1 inch. Any block maker can screen out a sample of his aggregate through standard screens and carefully determine the percentages of various sized grains present and compare these with the theoretical percentages, and if he finds that he has too much coarse material, he can screen out his coarse aggregate and recombine it with the fine in the proper proportions to correspond with the parabolic curve. He can do likewise if he has too much of the fine. This process is called rectifying the aggregates. I am afraid that some think that if an aggregate is graded to the parabolic curve that it will produce the strongest concrete with, say, 12 per cent. cement and no stronger if 20 per cent. cement be used. This is not true, for the strength will be increased with the increased amount of cement, but for any given amount of cement the strongest concrete will be from that aggregate whose mechanical analysis curve conforms very closely to a parabola. This probably holds true for concrete whose largest stone is ½ inch in diameter or larger. For mortar below number five mesh I think such an aggregate would have too much fine material. I have found the best result with a mixture of very coarse sand and a sand which is quite fine. (See Table 1).

I have dealt with this feature at greater length than I intended, but I feel that too much will never be said about the proper grading of materials for concrete blocks. I have seen too many blocks made with sand passing a No. 16 screen. The result was that the blocks were crumbly, soft and a disgrace to the industry. What wonders the addition of a little coarse gravel would have accomplished! I have also seen some men insist on using fine stone screenings, the bulk of which was a fine flour with a little stone about 1/16 inch to ⅛ inch in diameter, simply because they heard that crushed stone produced a stronger concrete than anything else. It is our solemn duty to

TABLE I.

This Table shows the percentages by weight of any size aggregate required for an ideal mixture, the largest aggregate of which corresponds with one of the sizes given below. For example, if the largest stone is 1″ in diameter the % of each of the other sizes is found in Column 3. The material smaller than .015″ includes the cement. This data is figured by the formula $P = 100\sqrt{\frac{d}{D}}$

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
DIAMETER OF STONE IN INCHES.	smaller than diameter to left.	between dia. to left and next larger dia.	smaller than diameter to left.	between dia. to left and next larger dia.	smaller than diameter to left.	between dia. to left and next larger dia.	smaller than diameter to left.	between dia. to left and next larger dia.	smaller than diameter to left.	between dia. to left and next larger dia.	smaller than diameter to left.	between dia. to left and next larger dia.	smaller than diameter to left.	between dia. to left and next larger dia.	smaller than diameter to left.	between dia. to left and next larger dia.
1.00	100.0	0.0														
.75	86.5	13.5	100.0													
.50	70.6	15.9	81.6	18.4	100.0											
.30	54.7	15.9	63.3	18.3	77.5	22.5	100.0									
.25	49.9	4.8	57.7	5.6	70.6	6.9	91.4	8.6	100.0							
.20	44.7	5.2	51.6	6.1	63.3	7.3	81.6	9.8	89.5	10.5	100.0					
.15	38.7	6.0	44.7	6.9	54.8	8.5	70.6	11.0	77.4	12.1	86.5	13.5	100.0			
.10	31.6	7.1	36.5	8.2	41.7	10.1	57.7	12.9	63.2	14.2	70.6	15.9	81.6	18.4	100.0	
.05	22.3	9.3	25.8	10.7	31.6	13.1	40.8	16.9	44.7	18.5	50.0	20.6	57.8	23.9	70.6	29.4
.02	14.1	8.2	16.4	9.4	20.0	11.6	25.8	15.0	28.3	16.4	31.6	18.4	36.5	21.2	45.7	24.9
.015	12.2	1.9	14.2	2.2	17.6	2.4	22.3	3.5	24.5	3.8	27.4	4.2	31.6	4.9	38.7	7.0
Small'r than .015	......	12.2	......	14.2	......	17.6	......	22.3	......	24.5	......	27.4	......	31.6	......	38.7
TOTAL.....	......	100.0	......	100.0	......	100.0	......	100.0	......	100.0	......	100.0	......	100.0	......	100.0

disseminate broadcast full information on the subject, so that no man can have any excuse for using too fine an aggregate.

The selection of the aggregates and the proper proportions properly taken care of, the next important consideration is that of properly mixing the concrete. The superiority of machine mixing over hand mixing is an accepted fact and needs no argument. Mixers should be used by all means, and none but power mixers. Hand mixers are man-killers, as anyone will testify who has used one. In choosing a mixer for concrete block work great care should be taken that it will mix semi-wet concrete and will handle either dry or damp materials. Every mixer will not do this, neither will every mixer that is satisfactory for a wet concrete prove equally so for concrete used in blocks. It is an easy matter to mix concrete of the consistency commonly used in monolithic work, but that used in blocks, being drier, is far more difficult to mix.

FACING.

As nearly all concrete blocks are faced, considerable attention should be given this feature in their manufacture. Most blocks are sold on their appearance, so it is essential that the face be very hard and all corners and edges solid. For this reason it is necessary to use richer proportions for the facing than is used in the backing of the blocks. The usual proportions vary from 1½ to 3 parts of aggregate to 1 part of cement. When ordinary sand is used it should be graded from ⅛ inch down to No. 50 mesh, with no material smaller than No. 100 mesh. The proportions can be as low as one part of cement to three parts of sand. But in white facing the proportions must be richer, as all available white aggregates are either all one size, as in the case of white sand, or contain too much fine material, as is the case in most crushed white marble. With some white sand and crushed white marble I obtained very satisfactory results with three parts white cement, two parts crushed marble and four parts white sand.

In colored facing none but mineral colors should be used. With proper materials and care, I have been able to use as high as 50 per cent. coloring in the proportions of one part cement, one-half part color, and one part aggregate. Dark, deep shades cannot be obtained except with a large percentage of coloring. Colored blocks and brick can be brought to a high polish by grinding. This process exposes the aggregate and produces a very attractive appearance.

A recent innovation is that of the granite facing. This is becoming very popular and is being used in large cities for public buildings for which the ordinary block would not be considered under any circumstances. I have found the best proportions for granite facing are one part of cement to two and one-half parts of granite. The granite should pass a ⅛ inch screen and be graded to about 1/32 inch. These four facings comprise the principal facings used.

Process of Manufacture.

A few years ago a subject of considerable discussion amongst machine manufacturers was that of processes of making blocks. Each claimed that his was the only machine in which a wet block could be made, and that no other was suitable for that purpose. The facts of the case are that as wet a block can be made on one machine as any other. I find that the limit is reached when the concrete is too wet to bear up its own weight. By this I mean that beyond a certain consistency the block will settle, and instead of being 7¾ inches high, it may be 7¼ inches in height. This fact, then, determines the maximum moisture that can be used in concrete for machine-made products and will be the same for all makes of machines. It is evident, therefore, that there are but two processes for making concrete products—the poured or cast process, and the molded or so-called dry process.

In the poured process the concrete is of such a consistency that it can be poured into molds which may be of sand, iron, plaster of paris, or gelatin. There is no question but that the product made in sand molds is very beautiful, but its cost is so excessive as to be almost prohibitive. In some instances it was but slightly lower than cut stone. This fact is its greatest drawback and accounts for its decline in favor of the molded concrete block.

In the molded process the concrete is made as wet as is possible to still permit of molding. It is then tamped or pressed into molds and usually removed immediately. The mold is thus used over and over again. This reduces the cost of the block to a minimum and enables the block manufacturer to compete with other building materials. Molded concrete is now being made as good as cast stone. This fact, combined with low cost, is bringing the concrete block to the front as one of the best building materials of the day. There has been considerable discussion as to the proper amount of water to use in concrete blocks. Most blocks are used when they are about 28 days old, so it is of the greatest importance to have the blocks as strong as possible at this age. Therefore, the consistency producing the strongest blocks at 28 days is the best. Actual experiments show that blocks which show the highest crushing strength at 28 days are those in which so much water has been used that it will flush to the surface when a pile of the concrete is trowelled with a steel trowel or shovel two or three times. This amounts to from 8 to 10 or 11 per cent. of the total weight of the dry materials. Concrete of this consistency can be used in any block machine. When made wetter the concrete settles and is not as strong. Concrete of this consistency is being used by the largest concrete block plants of the day. The well-known firm of Geo. Rackles & Sons, Cleveland, Ohio, make all their concrete of this consistency and mold all their stone in machines and special plaster, wood or gelatin molds. Their work is approved by architects everywhere and is the best argument in favor of the mold or so-called dry process of block making. Mr. Barber is using the same process in making blocks for all his buildings at Barberton, Ohio.

WATERPROOFING.

Many claim that the block made by the wet process is more waterproof, but I have seen blocks made wet in the molded process which were so waterproof that water would stand on same for a considerable time before being absorbed. Neither process will make concrete absolutely waterproof or nearly so. There are a large number of people who claim that a block with low absorption is all that is needed for practical purposes. This is true to a certain extent, but we must not forget that the average builder who is putting up a fine residence will not consider concrete blocks unless the manufacturer thereof will guarantee them to be absolutely waterproof, or very nearly so, absorbing, say, not over one-half per cent. of moisture. I have done considerable experimenting, trying to produce such blocks without the use of waterproofing, but have failed to do so. I do not think that it is possible to reduce the absorption below three per cent. without the use of a waterproofing. By the use of a waterproofing I have reduced it to 27/100 of 1 per cent., and I think that this absorption was merely due to the surface water on the specimen, so that the block itself probably absorbed no water whatever.

I will admit that with proper care and graded materials concrete blocks can be made reasonably waterproof, but I contend that they are not waterproof enough, in that they retain moisture too long. We are all familiar with the looks of the average concrete block house after a rainstorm. It usually remains damp and dark looking for two or three days, while one that is waterproof dries off in a few hours. Any man would prefer to live in a house that looked as dry a few hours after a rainstorm as before, rather than live in a building that remained a dull slate color for several days, so we should endeavor to turn out blocks that will give these results.

As long as blocks cannot be made as waterproof as this without a waterproofing, I will be in favor of the use of the proper waterproofing. I know well enough that a great number of waterproofings do not produce waterproof concrete, but there are also a number that do. It will be found that concrete blocks are made more impervious to moisture and more dense by machine tamping.

In testing a waterproofing, full-sized blocks should be made with same. Only such will be a reliable guide. Tests on small cubes are worthless, as such specimens cannot be tamped nearly as thoroughly as a full-sized block, and consequently will not be as dense and waterproof. I have often noticed that waterproofing is confused with permeability. A block may be waterproof and yet be very permeable, and vice versa. Waterproofing refers to the per cent. of water absorbed by a dry specimen compared to the dry weight of same, whereas permeability refers to the amount of water that will pass through the same specimen in a given time, when placed under a pressure of water.

It is evident that the concrete block used above grade need not be impermeable, but should be waterproof. A waterproofing may not produce an impermeable block and yet make a waterproof one.

In making white face concrete blocks it is absolutely necessary to use waterproofing, as the white aggregates that are most common consist of white sand, which is very fine, and crushed marble or stone, none of which will produce waterproof concrete, the first because it is uniform in size and the second because it has too much flour.

Even assuming that concrete can be made waterproof with proper materials, the average man cannot get these, and must make use of what he has, so it seems to me that the sooner the use of a good waterproofing becomes general, the sooner will concrete building blocks be used to a greater extent and the sooner they will overcome the objections of their present severest critics.

Curing.

When the questions of aggregates, proportions and process of manufacture and waterproofing are settled, the next feature is oftentimes neglected more than any other. The usual procedure is to sprinkle the blocks with water as soon as there is no danger of washing them away. When this is done the blocks should be kept moist constantly for a period of at least two weeks, but preferably four weeks. The temperature should be kept above the freezing point. There is no danger in keeping it too high, providing the blocks are kept wet constantly. If they are allowed to dry out, the hardening process is hindered. For this reason it is customary to keep the blocks under roof for three or four days at least, as it is much easier to keep them moistened under roof than outdoors. This method is extremely simple, so much so that it is surprising how few really apply it fully. I wish to emphasize the three essentials of this method: Commence sprinkling the blocks with a fine spray of water just as soon as you can do so without washing away the cement; keep the temperature comfortable, say 60 degrees F. or more; lastly, keep the blocks moist constantly for three or four weeks if possible. Many are doubtless mislead by the fact that blocks which are allowed to set until they show signs of drying out, seem harder than those that were sprinkled as soon as they could stand the water. This is true at first, but after a few weeks the second block will be firmer than the first and have clean, hard, sharp-cut edges.

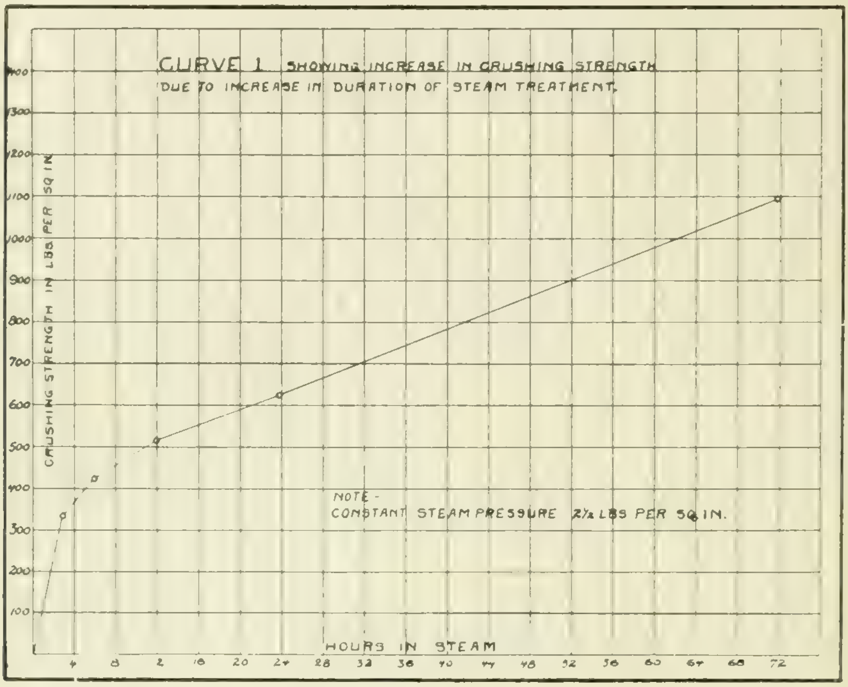
It is conceded by all who are familiar with the hardening of Portland cement that it requires both heat and water to properly harden same. It is at once apparent that the ideal method of curing must, therefore, combine both of these features. The only thing that fulfills these conditions is an atmosphere of steam. We find many successful exhaust steam curing plants to-day, but in nearly all of these the temperature is kept below 150 degrees F.

Some five years ago, together with Mr. R. J. Wig, after having given this subject careful study, I felt confident that high pressure saturated steam, which is also at a high temperature, would give the best results. If exhaust steam accelerates the hardening of the cement it is but a step further to assume that high pressure steam will do so even more rapidly.

We put this theory to a thorough test and investigated both the effect on the crushing strength of concrete of variations in the steam pressure and also the duration of the steam treatment. We found that the crushing strength was increased directly with the duration of the steam treatment, and also directly with the increase in the steam pressure. Concrete blocks cured in high pressure steam will be doubled in strength over those cured by sprinkling. This means that in order to get the same strength in the block cured in high pressure steam, only one-half the cement need be used. This probably holds true up to the point where the concrete would have so little strength when green on account of lack of cement, that it would crumble under its own weight.

I do know for a certainty that blocks can be made of one part cement to eight parts of coarse sand, which, when cured under high pressure steam, will crush at over 2,000 lbs. per square inch of area.

A second advantage of high pressure steam curing is that blocks so cured are ready for the market in two days. A third advantage is that rush orders can be filled as rapidly as the blocks can be made, and without danger of breaking the edges of the blocks, which happens so often in handling them when they are but a few days old and cured by sprinkling.



A very important advantage is that blocks faced with white aggregate and ordinary gray cement, when so cured, will be practically as white as white face blocks. This is amply attested to by the samples which I have here.

An ordinary block made of one part cement and four parts sand will crush at about 1,800 lbs. at six months, whereas the same concrete when cured in high pressure steam will crush as high as 4,900 lbs. per square inch, thus showing over twice the strength.

A 1-8 mixture, cured in high pressure steam, crushes as high as 2,100 lbs. per square inch, practically as high as the 1-4 air cured concrete at six months. Since nearly all blocks are used inside of 28 days, we are concerned with their strength at this age. Most building ordinances require a crushing strength of 1,500 lbs. per square inch of net area of the blocks. If a 1-8 high pressure steam cured block will crush at 2,100 lbs., it stands to reason that the block maker need not use a 1-4 mixture, as he must use in the air cured blocks. He will, therefore, save about one-half of the cement.

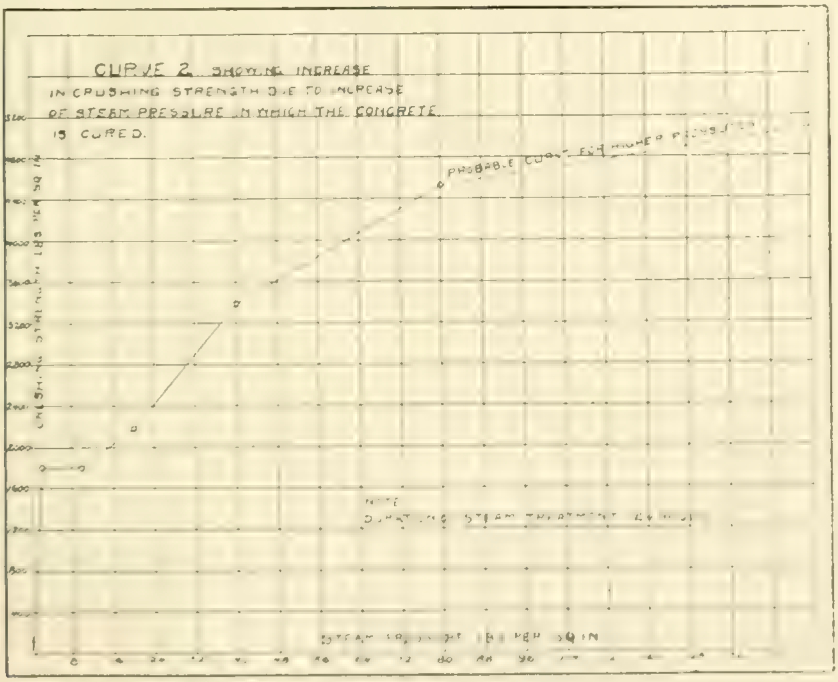
The tables which follow, and the curves drawn for same, show the increase in strength caused by an increase in the steam pressure and also an increase in the duration of same. The tables are practically the same as those given in the July, 1910, issue of Cement Era. (See Tables 2, 3, 4 and 5, and Curves 1 and 2).

PLANT ARRANGEMENT.

When a manufacturer has decided upon the above essentials, his next problem is that of arranging his plant. Every man will have his own views about this, but the following points should always be borne in mind when laying out any fair-sized concrete block plant.

The materials should be handled just as little as possible, and the best way to do this is to have the raw materials enter the plant at one end and the finished product should leave the plant at another point, either to one side or directly opposite where the raw materials enter.

Whether the aggregate is hauled from a distance to the plant or comes directly from a sandpit at the plant, it should be stored in one end of the building, and if it is to be graded, it should be fed into a bucket conveyor which will elevate it to the overhead screens, where the aggregate is then screened out into various sizes as desired and stored in overhead bins.



If the plant is not large enough to make it a paying proposition to put in screening machinery, the materials should be

TABLE II.

This Table shows the effect of the duration of steam treatment on the crushing strength of concrete. Proportions of the concrete are: 1 part Portland cement, 3 parts sand, and 4½ parts gravel.

Curing Treatment.	Crushing Strength in lbs. per Sq. Inch In 2 and 3 Days.	In 7 Days.
Not steam cured, but sprinkled, for 1 week	....	379
Steam cured at 2½ lbs. steam pressure for 3 hours	334	341
" " " " " " 6 "	427	484
" " " " " " 12 "	517	750
" " " " " " 24 "	627	660
" " " " " " 72 "	1,095	1,167

TABLE III.

This Table shows that the crushing strength obtained by steam curing is permanent. Proportions of the concrete are: 1 part Portland cement, 3 parts sand and 4½ parts gravel.

Curing Treatment.	Crushing Strength in lbs. per Square Inch In 2 and 3 Days.	7 Days.	28 Days.	3 Months.
Not steam cured, but sprinkled, for 1 week	..	379	1,068	1,426
Steam cured at 2½ lbs. pressure for 24 hours	627	660	1,382	1,488
" " " " " 72 "	1,095	1,167	1,270	1,522

TABLE IV.

This Table shows the increase in crushing strength of concrete caused by the increase of steam pressure in which same is cured. Proportions: 1 part Portland cement to 4 parts sand.

	Steam Cured at 2 lbs. for 24 hours.	10 lbs. for 24 hours.	20 lbs. for 24 hours.
Crushing strength in lbs. per square inch at 2 days	1,815	1,800	2,184

	Steam Cured at 40 lbs. for 24 hours.	80 lbs. for 24 hours.	80 lbs. for 12 hours.
Crushing strength in lbs. per square inch at 2 days	3,396	4,520	2,540

TABLE V.

Proportions: 1 part Portland cement to 4 parts sand. (All these specimens were made same as those given in Table IV., but were cured by sprinkling instead of steaming.)

	Age When Tested 28 Days.	3 Months.	6 Months.
Crushing strength in lbs. per square inch	1,851	2,286	2,343

stored at one end of the plant just the same and fed from here directly into the mixer. From the mixer it should be hauled in a dump cart and discharged at the side or back of the block machines, so as to be as handy as possible to the operator. The block machine should be located as closely as possible to the mixer, so as to avoid useless carting of the concrete.

One man, with a system of this kind, can easily keep 10 or 12 machines busy, whereas, without this arrangement, it often necessitates the services of one mixer man to keep one or two block machines busy.

The dump cart should preferably be elevated on a track, which arrangement makes it much simpler for the mixer man.

The block machine operator should use a shovel large enough to fill up the first part of the mold with one shovelful of concrete, and the second part with one or two shovelsful of concrete. There is no excuse for any block maker using four or five shovelsful of concrete to a block when he can do the same with two or three at the most.

The blocks should be carried as short a distance as possible from the place in which they are made. The best arrangement is to have racks made by two by fours, placed directly in front of the block machine, leaving a space of three or four feet between the block machine and the racks. The usual length of such racks is from 16 to 20 feet.

A 20-foot rack will easily accommodate twenty-six 8 x 8 x 16 inch blocks, allowing nine inches to each block. Four such racks can be piled one on top of the other and will accommodate 104 blocks. Four rows arranged like this will accommodate over 400 8 x 8 x 16 inch blocks.

With automatic tamping machinery and materials provided as above, one man will turn out regularly from 400 to 450 8 x 8 x 16 inch common blocks per day, or 350 to 400 faced blocks per day. All these are piled in a space about 20 feet long and not over seven feet in width. If exhaust steam is used for curing the blocks the curing room should be so arranged as to accommodate this number of blocks and partitioned off so that one set of blocks may be curing while another room is being filled with finished blocks.

Much has been said about the use of cars for racking concrete blocks. It is no doubt true that many manufacturers have found them to be a great advantage. On the other hand, there are many large plants in which the arrangement is so simple that it is difficult to ascertain whether he use of cars would simplify matters or not. I have in min dthe concrete block plant of the Menefee Art Stone Co., at Fort Wayne, Ind. Their arrangement is approximately what I have described above, and their men are actually turning out this number of blocks day after day.

After the blocks are cured two or three days they can be piled up on trucks and removed to the storage yard, where they should be sprinkled for a period of three to four weeks as described above.

Almost every concrete block manufacturer will manufacture a considerable amount of ornamental stone, as well as caps, cornices, etc. In order to avoid confusion and also to produce this at the lowest possible cost, it is best to provide a separate place for the manufacture of these articles, entirely independent of the space used for making the regular blocks.

An overhead trolley should be provided for handling heavy stone. This will also be found a great convenience in loading the stone on to the wagons, as one to two men can easily handle the largest sills and caps with a hoist far better than a half a dozen men can handle same by hand, and with much less danger of breaking them.

Too much attention cannot be given to plant arrangement. A careful layout will enable the manufacturer to produce blocks at the lowest possible cost, and hence increase his profits.

Cost of Concrete Blocks.

The cost of concrete blocks is always a very important consideration, especially when it is necessary to compete with other building materials.

In a general address of this character, however, it is impossible to cite cost data that will be applicable to any part of the country, as the prices of cement and aggregates vary so greatly.

The accompanying table, however, will be found of great value in estimating the cost of the materials and labor, but will not include any overhead expenses, such as the salaries of the officers of the company and that of a salesman for disposing of the product. It will be noticed that the data as to the output per man on different sized blocks is figured on hand labor. Wherever a plant is equipped with power tampers, mixers, etc., the output per man will almost be trebled; in fact, in many cases it has been.

Table Showing Quantities of Sand and Cement Required per 100 Blocks in Sizes Given Below:

SIZE OF BLOCKS	1-4		1-5		1-6			
	Yards Sand	Barrels backing Cement	Yards Sand	Barrels backing Cement	Yards Sand	Barrels backing Cement	Barrels facing Cement	Output per day per man by hand labor
8 x 4 x 16″ solid..	1.05	1.78	1.10	1.56	1.16	1.44	.375	135
8 x 4 x 16″ hollow	.81	1.37	.85	1.21	.89	1.10	.375	172
8 x 6 x 16″ hollow	1.11	1.88	1.17	1.66	1.22	1.51	.375	125
8 x 8 x 16″ hollow	1.41	2.38	1.49	2.12	1.55	1.92	.375	100
8x10x16″ hollow	1.90	3.21	2.00	2.84	2.09	2.59	.375	74
8x12x16″ hollow	2.27	3.84	2.38	3.38	2.50	3.10	.375	61
8 x 4 x 24″ solid..	1.58	2.67	1.66	2.35	1.74	2.16	.562	89
8 x 4 x 24″ hollow	1.10	1.86	1.16	1.64	1.21	1.50	.562	128
8 x 6 x 24″ hollow	1.84	3.12	1.94	2.75	2.02	2.50	.562	78
8 x 8 x 24″ hollow	2.19	3.70	2.31	3.28	2.41	2.98	.562	60
8x10x24″ hollow	2.73	4.62	2.88	4.09	3.00	3.72	.562	48
8x12x24″ hollow	3.27	5.53	3.45	4.90	3.60	4.46	.562	41

In general any block manufacturer should make a gross profit of 5c per 8 x 8 x 16 inch block. By this I mean the profit above the cost of the materials and the actual labor of producing, curing and storing same. Out of this gross profit of 5c per block must be paid the salaries of the salesman and officers of the company. It is, therefore, evident that where the output of a plant is low, that there must not be many officers or salesmen. A plant producing 150,000 8 x 8 x 16 inch blocks a year should have a gross profit of $7,500. The salary of a salesman, who would in this case probably be the only officer of the company, may be $1,800 a year. The expense of additional office help may amount to $1,200 a year, making a total of $3,000. The net profit will therefore be in the neighborhood of $4,500 per year. When one considers that the actual investment in a plant of this kind, including working capital, real estate and machinery, does not run over from $5,000 to $7,000, it will be seen that the concrete block business is very profitable. There are many plants making from four to five times this amount, year after year.

Strength of Concrete Blocks.

The use of concrete blocks has been restricted to a large extent in large cities on account of the fact that many city ordinances limit the use of eight-inch blocks to buildings of but one storey in height. This is probably due to the fact that the concrete block is a comparatively new building material and the average city authorities do not know much about the strength of same, and in order to properly protect their citizens against any inferior and untried building materials, they adopt measures which seem very harsh to those who are in a position to know about the strength of this splendid building material. There is no reason why concrete blocks should not be as strong as the concrete used in any monolithic work, and yet we hear very little about the danger of monolithic concrete collapsing.

Reinforced concrete is used to-day in skyscrapers and all important buildings, for which purpose it has proved itself to be without a peer. The concrete building block is merely one peculiar form of this material, so in this respect it is really a tried and proven material.

In some cities where this question has been investigated ordinances have been adopted requiring a crushing strength at 28 days of 1,000 lbs. per square inch of gross cross sectional area of the block as it is used in the building.

Assuming a factor of safety of five, which is greater than that used in steel construction, and which is an average used in other work, the allowable stress will be 200 lbs. per square inch of gross cross sectional area.

The distance of outside bearing walls of an average dwelling will not be over 25 ft., nor will it be greater than this in the average store building. If the distance is any greater, then supporting pillars will be found between the bearing walls.

As an example, let us suppose that the distance between outside bearing walls is 25 feet, and that the total floor loads, including the live load and the weight of the floor, is 110 lbs. per square foot, and that the roof load, including the weight of the roof, snow and wind loads is 57 lbs. per square foot. These loadings are probably higher than what is found in the average building. One 8 x 8 x 16 inch block weighs about 50 lbs., or 38 lbs. per lineal foot. Per square foot of wall the weight of the wall will run about 57 lbs., therefore the wall weight per storey per lineal foot of wall will be 12 x 57, or 684 lbs.

For the sake of simplicity let us consider a strip between two such bearing walls, one foot long and 25 feet wide, or an area of 25 square feet. The floor loads, etc., will be carried by one foot of each side wall. The floor load per storey per lineal of the two side walls is 25 x 110 lbs., or 2,750 lbs. The floor load per storey per foot of each side wall is one-half this amount, or 1,375 lbs.

The weight of the wall per lineal foot of each side wall is 684 lbs., making a total load per foot of each side wall of 2,059 lbs. per storey. An 8 x 8 x 16 inch block is 128 square inches in cross sectional area. Assuming a safe loading of 200 lbs. per square inch and a factor safety of five, the allowable load on one 8 x 8 x 16 inch block will be 25,600 lbs. Per foot of wall this amounts to 19,200 lbs. This would allow 9 1/3 twelve-foot storeys to be carried by an eight-inch concrete block wall, loaded as above.

It must also be remembered that a concrete block crushing at 1,000 lbs. per square inch at 28 days, will probably have a crushing strength of 1,200 lbs. per square inch at six months, so that the factor of safety at six months will be six instead of five. In view of these facts it is hard to understand why any city authorities should limit the use of an eight-inch concrete block to one-storey buildings only. They should at least be allowed in two-storey buildings, and if necessary the basement wall in such buildings could be made of 10-inch blocks. On this basis the actual factor of safety would be 4 x 5, or 20, instead of an apparent value of 5.

It is true that some portions of a building will be loaded more heavily than I figured above on account of openings in the wall, but the openings are not likely to run over 25 per cent. to 33 per cent. on the wall area, so the actual factor of safety in any part of the building is likely to be at least 12 where it is allowable to use an eight-inch wall for two storeys.

In steel work a safety factor of more than four is seldom required, so it certainly seems to me that any city authorities would be on the safe side if they allowed the use of eight-inch blocks in two-storey buildings, for they can safely figure that the actual safety factor in such cases will be 12, and this will continue to increase, for the concrete will increase in strength the older it gets.

We should not feel that city authorities discriminate against the use of concrete blocks, but rather that the associations of cement users are at fault in not paying more attention to the drawing up of proper ordinances which can be presented to the various city authorities, and I heartily recommend, if this Association has not already done so, that it take steps to draw up the proper ordinances that will be fair both to the block manufacturer and the user thereof, and figure the allowable thickness of walls on a basis of the crushing strength of the blocks, and also limit the maximum loading to which any part of the wall may be subjected. Such ordinances should cover the method of manufacture as well.

It will be found that no ordinance can cover all buildings, especially factories and warehouses. In such cases the size of the pilasters and bearing walls must be figured according to the actual load to be carried by same.

While I do not expect all my hearers to agree with my views, I sincerely hope that some of the points that I have brought out have proved of interest.

And now a note of warning and a message of hope to the people generally. I quote the words of Mr. Franklin H. Wentworth, of Boston, who in addressing the Fire Underwriters' Association of the Northwest, at the Hotel La Salle, Chicago, recently, said:

"That man of the street still believes that the cost of fire comes out of the coffers of the insurance company. If we could go around to the people each year and say to the head of the family, 'Here, give me your check for $15 for your share in the national carelessness,' it might make an impression upon his mind. He does not seem to realize that everything he eats, everything he wears, everything he puts into his house, bears the fire tax. We have got to bore this into his thick skull and make him understand it.

"The merchants are beginning to understand the situation, but the man of the street has no idea what it means, and so he goes around throwing his matches here and there in utter carelessness."

Do not these words breathe to you the unbounded possibilities of the future of the concrete products business? Does it not say to you in so many words, "People must build of fire-resisting materials." In the United States, and here in Canada, the slogan has gone forth, "We have got to teach the people to build better." Does anyone doubt, therefore, that those entering into the business of making concrete products can do else than make an assured success, if proper business methods are adopted and a standard of honesty be adopted?

If my closest relative, my dearest friend, would ask me if I knew of a business chance, I would unhesitatingly advise them to enter the business of making concrete stone in its every variety. Across the line thousands are in the business, thousands have been wiped out, because they were dishonest or inexperienced, and in all business it means the survival of the fittest. In every hamlet, every town, every city, plants are starting, and already the demand for concrete building blocks is immeasurably greater than the supply. Good machinery and good common sense are the chief assets necessary.

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THE GRADING OF MATERIALS IN THE AGGREGATE.

BY H. P. BOWES.

In the following, I have endeavored to put in the form of a paper a few ideas on the subject of the grading of the materials in the aggregate for concrete work.

The firm with which I am connected, the Warren Bituminous Paving Co., have made a specialty of the grading of the materials used in the surface coat of roadways and have met with a great deal of success. The specifications for the grading which they have adopted call for the mineral aggregate being proportioned to size, so as to produce a mass having 21 per cent. or less of voids at the time of construction. They have found that any method of construction calling for the use of mineral aggregate will reach a degree of perfection proportional to the ability in so grading or arranging the mineral constituents as to reduce to a minimum the existing voids. Then with the addition of the proper mortar a mass will result which will not only be dense but will be nearly waterproof.

Upon the other hand, the farther these principles are receded from, the more certain failure is courted.

It would then seem that the paramount principle is the proper grading of the mineral constituents.

Perhaps this subject really applies more forcibly to reinforced concrete work, where great strength is required for the least possible weight, for large and important structures. It pays from an economic standpoint to make very thorough studies of the materials of the aggregates and their relative proportions. This fact I fear has been seriously overlooked in the past and thousands of dollars have sometimes been wasted on single jobs by neglecting laboratory studies or by errors in theory.

Since cement is always the most expensive ingredient, the reduction of its quantity which may very frequently be made by adjusting the proportions of the aggregate so as to use less cement and yet produce a concrete with the same density, strength and impermeability together with the reduction of size, is of the utmost importance. If gravel or stone having particles of uniform size is used, it must be recognized that the work will cost 10 per cent. or upwards more on account of the additional quantity of material required to make a given volume of concrete. In measuring the gravel or stone before mixing there will be less solid matter in the measure and, consequently, more sand and cement will be necessary to fill the spaces between the stones. This fact which is often overlooked even by experienced men is illustrated in a somewhat exaggerated fashion in Figures 1 and 2. The proportions of this particular mix are 1:2½:5.

Here Figure 1 illustrates the measurement of the dry materials and the mixture produced therefrom when the stone has been screened to one uniform size, while Figure 2 shows the dry materials and the mixture when the stone has been graded somewhat.

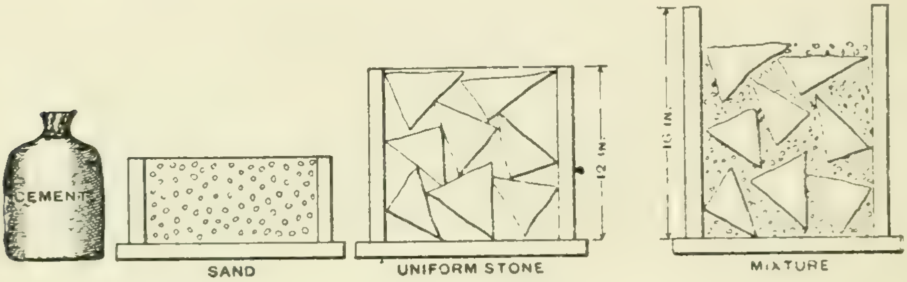

FIG. 1—Diagram illustrating measurement of dry materials and mixture when broken stone is of uniform size.

It is obvious at a glance that the uniform stone measured in Figure 1 contains less solid stone than the graded stone measured in Figure 2. The spaces between the stones in the first case are very nearly equal to the volume of the solid particles and as the measure of sand is one-half that of the stone, and the particles of cement fill the voids in the stone, this sand and cement mixes in between the stones filling the spaces or voids and resulting in a mixture but very slightly greater in volume than the stone alone.

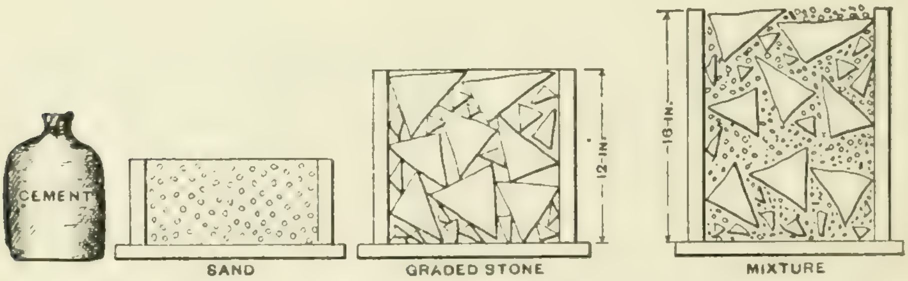

FIG. 2—Dry materials and mixture when the stone is of varying sizes.

In the second case, Figure 2, the spaces between the large stones in the stone measure are filled with graded smaller stones so that there is a much smaller volume of spaces or voids. Hence, when the sand and cement which are identical with that in Figure 1, are mixed with it the volume of the mixture becomes considerably larger than the original bulk of the stone. Consequently, if we are tied to definite proportions of materials more concrete will be made with graded stone than if the stone has been screened to a uniform size.

If, on the other hand, the proportions of the materials are changed on account of the fewer voids in the mixed stone and less sand and cement are used, a saving in these materials results.

PROPORTIONING BY VOID DETERMINATION.

The method often used in determining the voids in stone and sand by finding the quantity of water that can be poured into the voids of a unit measure of stone or sand and then the amount of sand or cement as the amount required to fill the voids in the stone or sand, is not satisfactory. The greatest inaccuracy of this method is due to the difference in compactness of the material under varied methods of handling, and to the fact that the actual volume of voids in a coarse material may not correspond to the quantity of sand required to fill the voids. For the grains of sand separate the stone and with most aggregate a portion of the sand is too coarse to get into the voids of the coarser material. That is, in a mass of graded stone many of the individual voids are so small that the larger grains of natural bank sand will not fit into them but will get between the stones and increase the bulk of the mass. This increase in bulk means that more sand is required than the actual volume of voids in the coarser material.

Perhaps the most practical way of obtaining the desired proportions after all is by

VOLUMETRIC PROPORTIONING BY TRIAL MIXTURES.

An accurate and simple method to do this is by trial batches. The apparatus is easily obtained at any time, consisting as it does of a scale and a cylinder which may be a piece of wrought iron pipe 10 to 12 inches in diameter capped at one end. Measure and weigh the cement and stone and water and mix on a piece of sheet steel or other non-absorbent material. The mixture having the constituency the same as intended to be used in the work. The mixture is placed in the cylinder carefully tamped and the height to which the pipe is filled is noted. The pipe should be weighed both before and after being filled so as to check the weight of the material. The cylinder is then emptied and cleaned.

Mix up another batch using the same amount of cement and water, slightly varying the ratio of the sand and stone, but having the same total weight as before. Note carefully the height in the cylinder again which will be a guide to the other batches to be mixed. Trials are made until a mixture is found that gives the least height in the cylinder and at the same time works well while mixing, all the stones being covered with mortar, and which makes a good appearance.

This method gives fairly good results but does not give the changes in the physical sizes of the sand and stone so as to secure the most economical composition, as would be shown by a thorough mechanical analysis.

Mr. A. E. Schutte in studying the proportions of materials for Bituminous Macadam Pavement for the Warren Bros. Company has very effectively developed the method of volumetric synthesis with dry materials. His experiments included various classes and sizes of stone, sand and screenings, ranging from three inches in diameter down to that which passes a 200 mesh sieve. He found that the best method for compacting dry materials such as sand, gravel or broken stone, is to place them in a vessel the shape of an inverted cone, with the largest diameter at the bottom. The cone is filled with the coarsest material and taken by a laborer who compacts it by repeatedly striking the cone against the ground, keeping the measure full by adding new material of the same kind. When it ceases to settle the contents are emptied and mixed with a portion of a finer material replaced in the measure and compacted as before.

By repeated trials the exact size and maximum volume of successive finer materials which may be added without appreciably increasing the bulk of the coarsest after thoroughly compacting are determined. Mr. Schutte has found that for different shapes of particles the proportions of each size must be varied, but having determined the required percentages for a certain stone, that is, for a stone from a certain quarry, the proportion of the sizes from day to day need be varied but little.

EFFECT OF GRADING AGGREGATE ON THE DENSITY OF CONCRETE.

The fact that the densest mixture is secured with particles of different sizes is very evident from the figures 3 and 3a, and needs no other proof. It follows that the least density and, hence, the greatest percentage of voids occurs when the grains are all of one size. The converse of this proposition, that the smallest percentage of voids occurs in a mixture graded so that the voids of each size are filled with the largest particles which will enter them, is also illustrated in figure 4. This is very important in its application to the selection of materials for concrete. The mass of this last mixture is evidently denser —contains more solid material. This relates directly to the difference between mortar and concrete. The substitution of stones for small masses of sand reduces the voids and consequently the quantity of cement required. Extending this principle to the fixing of the proportion of sand and stone it is evident that for maximum economy and equal strength there should be used the largest possible quantity of stone in proportion to the sand.

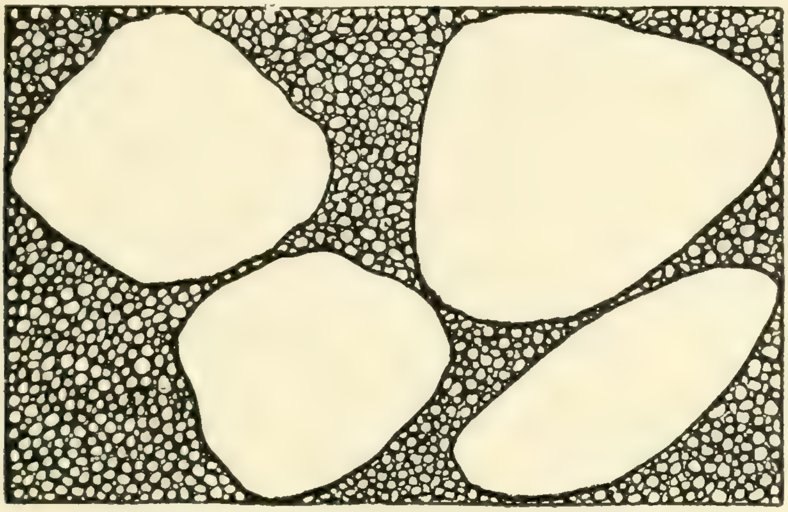

FIG. 3—Large stones with voids filled with sand.

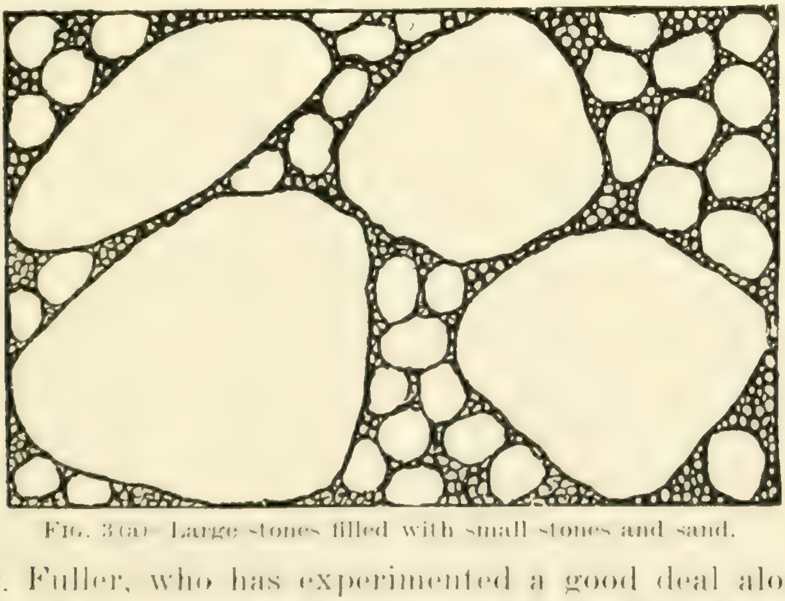

FIG. 3(a) Large stones filled with small stones and sand.

Mr. Fuller, who has experimented a good deal along this line, found that by grading the materials of his aggregate, figuring at a point where he only required one part of sand to five parts of stone, that for this mix the modulus of rupture was 504 pounds per square inch. As he increased the amount of sand the strength decreased; when he reached equal parts of sand and stone the modulus of rupture was 355 pounds per square inch, and when he had twice as much sand as stone the modulus of rupture was 210 pounds per square inch. The total amount of aggregate in each case was the same, namely, 1 part cement to 6 parts sand and stone.

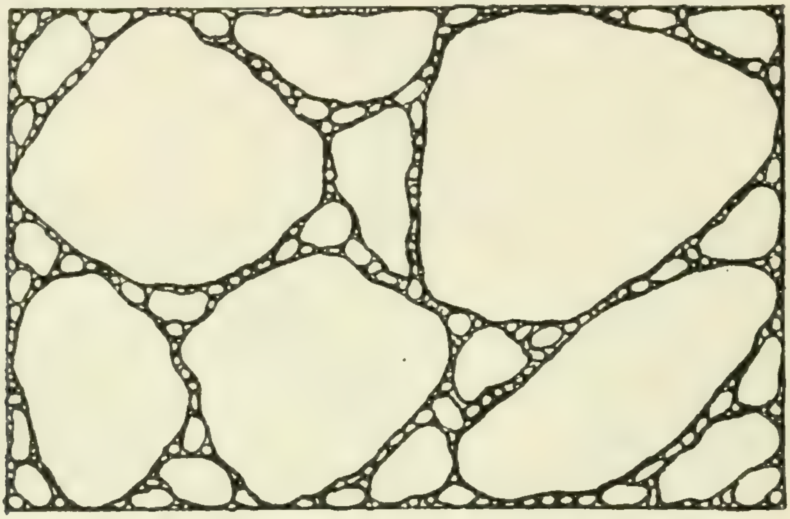

FIG. 4—Large stones with voids filled with medium sized stones surrounded by smaller stones and sand so as to give graded mixture.

In 1901 tests were made at the New Jersey Water Company's reservoir on the comparative strength of different proportions of concrete aggregate. These tests indicated that for the materials used there was a certain mixture of sizes of the aggregate which with a given percentage by weight of cement to the total aggregate, gave the highest breaking strength. In practice it was also found that the concrete made with this mixture worked more smoothly in placing. These tests led to a more extended series of experiments by Messrs. Sandford Thompson and Frederick Taylor at Jerome Park Reservoir, New York. The method of procedure and results of tests are given by Wm. B. Fuller and Sandford Thompson, Transactions of American Society Civil Engineers, Vol. 59, page 67, 1907. The mechanical analysis diagram of their tests furnished a ready means of studying the effect of various sized particles on the density of concrete. For this purpose crusher run stone and bank gravel were screened into twenty-one sizes, ranging from 3 inch down to that passing a 100 mesh sieve and over 400 tests were made with different combinations of the sizes of stones to obtain the proportions which would produce the densest concrete mixture. From these tests they derived the following laws:

(1) Aggregate in which parts have been specially graded in sizes so as to give, when water and cement are added, an artificial mixture of greatest density produces concrete of higher strength than mixtures of cement and natural materials in similar proportions. The average improvement in strength by artificial grading under the conditions of tests was about 14 per cent. Comparing the tests of strength of concrete having different percentages of cement it is found that for similar strength the best artificially graded aggregate would require about 12 per cent. less cement than like mixtures of ungraded materials.

(2) That the strength and density of concrete is affected but slightly, if at all, by decreasing the quantity of medium stone of the aggregate and increasing the quantity of coarsest stone. An excess of stone of medium size on the other hand appreciably decreases the density and strength of the concrete.

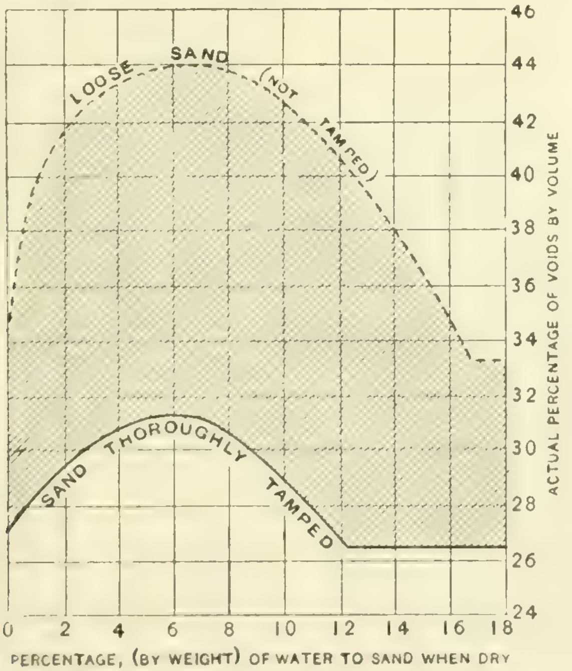

FIG. 5—Percentage of absolute voids in a natural bank sand, containing varying percentages of moisture.

Perhaps it might be worth while calling attention to the effect which dampness has on the volume of sand and stone screenings and stone, especially sand and stone screenings, and hence of the allowance necessary to make for this. The diagram, Figure 5, was plotted by Mr. Fuller from experiments from a single sample of natural sand mixed by weight, with varying percentages of water and illustrates the effects of moisture upon the actual percentage of voids in sands, loose and tamped. The volume produced by varying degrees of compacting are located between two curves. It is noticeable that both loose and tamped sands increase in volume with the addition of water and reach a maximum at about 6 per cent. of water, then decrease to finally return to slightly less than their original bulk. The same sand it may be seen may contain from 27 per cent. to 44 per cent. of absolute voids according to the percentage of water and degree of compacting. The percentage of water by weight which will give the greatest bulk corresponding of course to the largest percentage of voids, varies with different sands from 5 per cent. to 8 per cent.

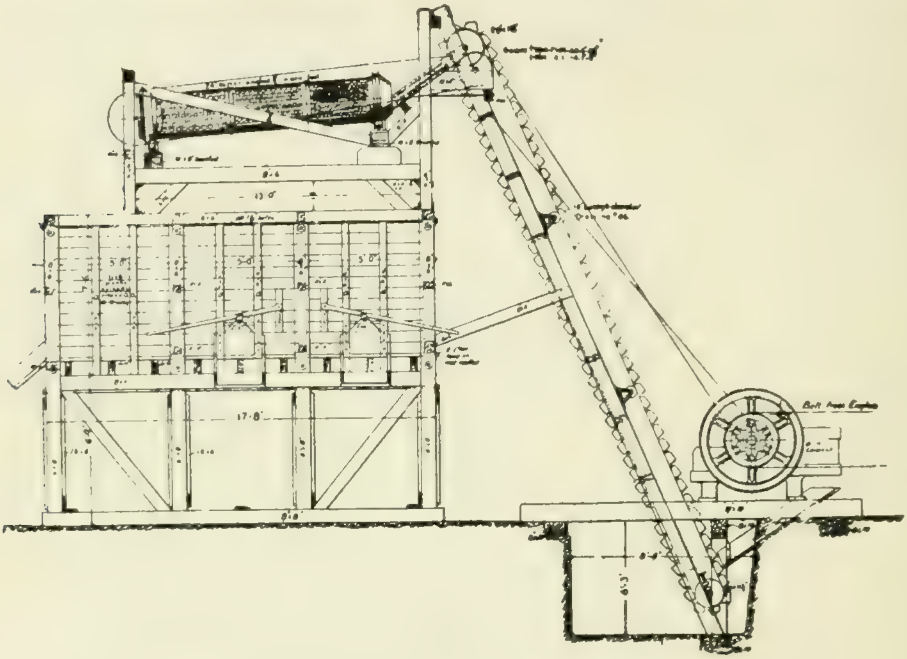

FIG. 6—Small crushing plant with elevator, screens and bins.

In figure 6 is shown an ordinary stone crushing outfit with a sectional screen of three different sizes of mesh. The screen used by the Warren Bros. Company is composed of one screen within the other, the stone is delivered into the end of the screen of course which contains the smallest openings. The inside screen of the first section would be about ¼ inch mesh and the outside about a ten mesh, and not quite so long as the ¼ inch section; so that the fine material passing the ten mesh would fall directly into a bin below and the ¼ inch be delivered from the end of the outside screen to another bin. Similarly with the other two sections, the second section would be about

¾ inch inside and ½ inch outside, and the third 1½ inch inside and 1 inch outside. This only permits, of course, of the grading of the stone into six different sizes.

The bins for receiving have a chute at the bottom and a slide to control the flow of the stone. Below these bins is a hopper set on a six beam scale so that the desired amount of each size of stone can be drawn into the hopper without changing the weights on the beams. Of course this method would not be applicable to concrete work unless the mixer was set directly below the hopper, for if the materials are loaded into wagons or carts and dumped in piles the large stones roll to the outside of the pile and very little good would result from the previous grading.

The only feasible manner to take care of the different sizes of stone if the screens are not near the work is to use bins to hold the different sizes of stone.

Figure 7 shows a sawed section of Bitulithic pavement taken from Palmerston Boulevard, Toronto. This shows the grading of the different sizes of stone in the pavement and the way in which they inter-mix with each other. The sizes of stone in this sample range all the way from 200 mesh material up to 1½ inch.

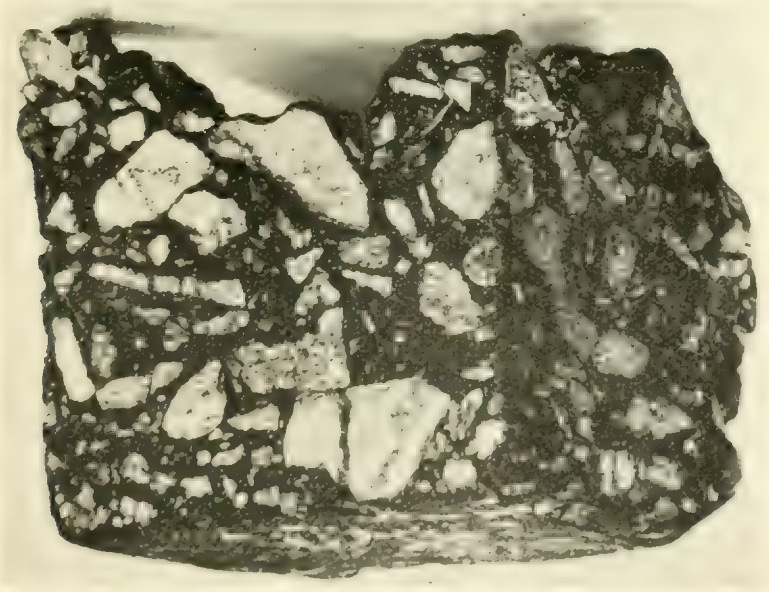

Fig. 7 Sawed section of bitulithic pavement.

Mr. Gillespie and the writer crushed two blocks of concrete. One was made of an aggregate of one inch stone and sand, the proportion being 1 cement to 3 of sand to 5 of stone. The total aggregate when dry weighing 30 pounds, to this was added 5 pounds of cement, then mixed with water and placed in a box 8 inches square. The other sample was made of a graded aggregate of the following proportions:

1 inch stone	3	lbs.
¾ inch stone	7	lbs.
½ inch stone	6	lbs.
¼ inch stone	5	lbs.
8 mesh material	½	lb.
16 mesh material	1½	lbs.
20 mesh material	1½	lbs.
50 mesh material	3	lbs.
80 mesh material	1	lb.
100 mesh material	1	lb.
200 mesh material	½	lb.
Total	30	lbs. as before.

This mixture was mixed with the same amount by weight of cement as the previous sample and put in the same size box. The two samples were kept side by side as they dried so that all conditions except the grading of the aggregate were identically the same. The blocks were only six days old when crushed which accounts for the comparatively low strength.

For the ungraded sample the load at point of failure was 560 lbs per square inch, and for the sample with graded aggregate the load at point of failure was 679 pounds per square inch, or a gain in strength by grading the aggregate of 21 per cent.

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THE WHITE AND THE GRAY.

BY J. MAXWELL CARRERE.

I wonder how many of you stop to think of the man, his struggles, anxieties and years of toils for your welfare, when in the course of a business day as a matter of fact you open your mail, brought to you by the locomotive or the steamer, when you receive a telegram and immediately reach for your 'phone, and rushing for the elevator, descend to the street, make for the subway, and being held up by excavations, you mentally, if not actually, rail at the contractor and his men handling that nasty, sloppy concoction you know is concrete?

What would have happened 100 years ago to you under the same conditions, granted that a letter was mailed only a few days ago from the Mother Country or the Province of Louisiana? Think that over!

HISTORICAL.

All events follow the course of a nation, and necessity creates achievements.

With the rapid growth of our country in the last century material development predominated, and the intellectual could not keep pace with the times, excepting in those communities—well populated from the first and located at the seaboard—where the continual influx from the Old Country kept pouring into the States in ever increasing numbers, and bringing with them, if not new ideas, at least new problems to be solved by the young nation. Problems of government, religion, education, hygiene—all subjects that required, not only the practical, but the learned man of these communities in their solution.

Our institutions of learning were few, and while endowed with enthusiasm of the learned early settler, were struggling under great difficulties, supported by the devoted love and loyalty of their faculties and early graduates.

The material development of the country had created great individual wealth, and with this wealth came the desire of the individual to make his progeny the greatest ever. And having found in the course of his own experience that whenever a knotty technical problem was involved the foreign engineer or professional man had solved it for him, it is most natural that his first idea was to send his sons abroad for that education he did not have himself. Later, through national pride, he wisely spent that very wealth to endow our institutions of learning, that they might even surpass similar institutions abroad, or at least to furnish that particular education he conceived young America needed. And thus began the rapid development and enlargement of our colleges, and towards the end of the 19th century with rapid strides we see colleges springing up in every direction, and the older colleges becoming universities.

With this period in our national history we also perceive the renaissance of art in America, and I say renaissance wittingly, for the Indian and the early settler had a conception of art. Building operations had increased at a tremendous pace; the architect was coming into his own; he was no longer a contractor, but a recognized authority. The engineer already recognized seemed to develop a desire to create the impossible. To span rivers he had to invent the steel cable; to take care of increased traffic he had to design the elevated roads, and when that traffic again increased he had to create the tube. To do all of which he no longer could rely on the Roman and natural cements, but had to create that product which made those structures possible—Portland cement.

Portland cement is the best product of its kind devised before the 20th century. It superseded the Roman, the Puzzolans and the Grappiers, and later the natural cements, because of its more perfect chemical structure, physical strength and economic value. It is the cement of to-day. The cements of yesterday have passed into history.

On behalf of the men engaged in our industry, it is my purpose to give you an account of our stewardship, and point out to you in what manner we hope to serve the present and future generation.

Portland cement and its predecessors, are foreign discoveries or products which have been adapted and developed by American engineers. Twenty years ago the production of American Portland cement was small, and the methods of production primitive, being copied from foreign countries where labor was cheap and fuel high, whereas the reverse conditions existed in the United States and England. An English engineer, to save labor, conceived the rotary kiln, a revolving cylinder in which to burn the product and save labor. American ingenuity perfected it, and further improved it by perfecting a method of burning powdered coal. To two engineers—one American and one an Englishman—Messrs. Seamans and Hurry, backed by strong financial assistance, do we owe the first real development that put Portland cement within reach of every man. That they have fared well can but be gratifying to any man who admires achievement, and, in fact, I will go further and state that the fundamental principles of the cement mill as laid down in the first plant designed by them at Northampton, Pa., remained such without alterations or improvements of a vital nature, till a few years ago, when the wizard of Menlo revolutionized the industry with his long kiln and grinding process. While both were decried by everyone at first, the long kiln has been universally adopted both here and abroad, with the reservation often expressed that there is nothing in it but volume.

The future will demonstrate the value of Mr. Edison's other conceptions. For my part I consider most of them admirable, but an engineer must always consider the economic side of the problem as well, and not having had the opportunity of doing so on his other conceptions, I can only say that they are fundamentally sound and that the industry to-day is in such a state of perfection that past great economies are no longer possible.

During all this period of less than twenty years, building operations increased at a tremendous pace, and Portland cement became the recognized structural cement. It is the cement of to-day. It is a grayish, sometimes a bluish dull material, of great structural value for the engineer, the architect and the builder, but its appearance precludes the possibility of its ever attracting the artist. There is hope, therefore, that the cement of to-day—the gray Portland cement—will remain the structural factor in the future. But the cement of to-morrow is the White Portland cement.

The White Portland is purely American, for while scientists knew that a white Portland cement could be produced from pure kaolins and pure limestone, and some companies tried it, the cost of such a product was commercially impossible. Practically, I consider it equally unattractive. After years of toil in experimenting, four of which were spent in the present plant, Allentown, Pennsylvania, gave birth to the White Portland cement of commerce.

The history of four years since the White Portland cement was introduced is most interesting, for they were years of distress and panic, years of curtailment of operations, years of economy. They are interesting because of the possibilities created for the gray by the White Portland cement, because of the artistic possibilities in a cement structure of to-morrow and because of the further artistic education it may create for the welfare of the general public.

In quality there is no difference between the gray and the white. In purity, color and adaptability the white is the darling. Like in many other paths of life, there is no occasion for jealousy, for being of one nature, one quality, one family, they can be made to harmonize ideally in attractive and durable construction, heretofore unattainable.

The White Portland cement destroyed the distinction between structural and ornamental when applied to cement—it no longer exists. The engineer can build his concrete bridge or his stack as artistically as he can conceive it of Portland cement and has no possible excuse for the creation of an unattractive structure.

Portland cement is plastic. White Portland cement is not only plastic, but admirably adapted for ornamental work, no matter how elaborate, and besides can be colored any tint to suit the particular artistic temperament of the individual, and for this reason I feel that this baby industry will not only greatly benefit the gray Portland cement industry, but the individual and the public in general, by putting within reach of all the attractive home and pleasant surroundings constructed throughout of indestructable material—Portland cement.

Our public to-day craves for the refined and the beautiful, and while in our youth as a nation we still demand the big, and sometimes the showy display of our conceptions, yet education, travel and association have raised the public standard in general to an appreciation of the fitness of things, to the difference between large and great, quantity and quality, and also to an appreciation that "in union there is strength" for the large and great, the quantity and the quality, the gray and the white, can be united to better serve our ends if properly assimilated.

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PREVENTION OF CORROSION IN METAL LATH.

BY CLARENCE W. NOBLE.

For the past year the writer has been investigating corrosion in steel and the methods of preventing it. The object of the investigation was to determine what improvement, if any, could be made in the protective coating used on Herringbone metal lath. The investigation brought to light much interesting technical information.

The nature and cause of corrosion was first studied. A very exhaustive investigation of the corrosion of iron and steel exposed to the atmosphere has been recently made by Dr. A. S. Cushman, of the U. S. Department of Agriculture. While his studies were confined primarily to the corrosion of fence wire, the results are of general value. He states that corrosion is primarily due to the presence in the air of minute quantities of carbonic acid and sulphurous gases. These acid gases are dissolved in falling rain and thus brought into contact with the iron. Their action then depends on the condition of the metal. If it is absolutely uniform in quality the attack is exceedingly slow and impotent. If, owing to the localized presence of impurities, different portions of the metal vary in electrical conductivity, then the acids and the varying portions of the metal form a miniature electric battery. A current is set up and that portion of the steel which is electro-positive as compared with the surrounding metal will be corroded. This explains why certain fence wires will corrode while others with the same exposure will last for many years. The local presence of impurities also explains why steam boilers will pit instead of corroding uniformly.

The action of wet plaster on metal lath depends on the nature of the plaster. Portland cement and lime plasters are strongly alkaline and will not allow the formation of acids in their presence. They therefore prevent the rusting of metal lath. The protection given by Portland cement is a permanent one. Lime plaster, however, has a greater attraction for moisture than Portland cement, and while lime protection may be permanent, still need is felt for further investigation before making a definite statement. Such investigation was not made, as the matter is a diversion from the subject in hand.

Plaster of Paris, which is the base of the many brands of patent and hard wall plasters, is actively corrosive. During the setting up process it has in a marked degree the tendency to start electric currents, and these currents cause the oxygen molecules which are momentarily freed by the chemical changes, to attack the steel. The corrosion is much more rapid than that due to atmospheric exposure. Iron rust, once started, works progressively, and the rusting in this case continues long after the chemical changes in the plaster, with their resulting electric currents, have ceased. It only requires the presence of a normal amount of atmospheric moisture to keep the action going.

The different types of protective coatings which were studied fall naturally into three separate classifications—paints, electrical insulators, and galvanizing coatings. Each of these works on a different theory. 1. The painted coating is merely an attempt to shield the steel from the plaster. If well done with an undiluted oil paint, it has some value. Linseed oil is subject to attack by lime and plaster of Paris, becoming what is known among painters as "dead." It is then porous and will not prevent corrosion. The first attack of the plaster is, however, expended on the paint instead of the steel, and for this reason the paint coating is of assistance. If the paint holds until the electric currents cease it has served a useful purpose; if not, it has at least delayed matters.

Metal lath should be coated with an exceptionally tenacious material. It is shipped in bundles and roughly handled during erection, and the coating is severely tested by scratches. In this respect even the best paint leaves much to be desired. Of course rust will immediately start at each scratch. If in the effort to reduce cost the manufacturer dilutes his paint with gasoline, the resulting coating is valueless, although it can be made cheap. Unfortunately most paint used for lath protection is thus diluted. Everything considered, the paint coating has the least to recommend it of any of the three types of protection considered.

The coating with electrical insulators proceeds on an entirely different theory. The attempt is here made to prevent the electric currents from reaching the lath. The cold japan coating used on Herringbone lath is of this class. It consists of an asphaltum varnish which is oxydized by the addition of a chemical dryer instead of by baking. It is proof against the action of all types of plaster and is a perfect insulator as long as the coating itself is perfect. The objection to it is that it is impossible to make it proof against the wear and tear of handling. If it is sufficiently elastic to prevent chipping, then it will scratch. There seems to be no neutral territory between these two difficulties. Its cost, five-eighths of a cent per square, is in its favor. While rust will start at the open scratches, the attack can never, as in the case with paint, be made through a "dead" coating. It will probably be long before it will be replaced for medium-priced work.

Galvanizing coatings work on still a different theory. The electric currents are here allowed to circulate at will, while their mischief is prevented. It has been previously stated that the currents flow only between metals which differ in electrical potential and that the attack is on that metal which is most strongly electro-positive. Now all zinc is more electro-positive than any commercial steel, regardless of its impurities. A coating of zinc, therefore, will cause the currents to run between the steel and the zinc instead of between the different portions of the steel, and the attack will be invariably taken by the zinc. Freedom from scratches in this method of protection is unessential. It is only detrimental when the steel is so far exposed as to disclose a localized impurity (a potential rust spot) entirely surrounded by exposed steel. In this case the rust spot would develop. So true is this that metal lath cut from galvanized sheets with all the raw edges exposed will show most excellent results.

In the search for a perfect protective coating only one type of paint, Marine Tocholith, was tried. This is not a mixture of oil and pigment, and is therefore, strictly speaking, not a paint at all, but as its action is similar to paint, it will be discussed here. It is a modified Portland cement. Its advantage is due to the fact that it has the property common to all types of Portland cement, of preventing rust and absorbing a small amount of rust already started. It grows harder with age and in time adheres very tenaciously to the steel. It was found that it would not attain sufficient hardness to stand bundling and shipping for several weeks. The manufacturers submitted a special sample designed to harden with unusual speed, but even this was not commercially feasible. It was rejected on account of its liability to scratching.

Two insulating coatings were considered, a baked and a cold japan coating. The cold japan is merely an improvement in the coating which has been used on Herringbone lath for several years. It was adopted to replace the old cold japan coating, but on account of the liability to scratching is not considered absolutely perfect coating. The baked enamel coating is used to a certain extent in the United States. It requires considerable expenditure for plant and only partially overcomes the scratching difficulty. When the lath is bent in forming cornice work the enamel is very apt to break. It is very expensive and is not regarded as sufficiently satisfactory to justify the cost.

A new metal called ingot iron was also considered. This is being used for the manufacture of Herringbone lath in the United States. The impurity which causes variation in electric potential in steel is manganese. A very slight variation in the percentage of manganese makes a wide variation in electric potential. As it seems to be impossible to secure an absolutely uniform distribution of manganese, the manufacturers of ingot iron have made a product in which this and practically all other impurities are omitted altogether. Electric currents are therefore not set up in ingot iron as the result of an acid bath, and corrosion is much slower. Acid tests with ingot iron show remarkable results. Tests with plaster were not sufficient to give a satisfactory verdict.

The metal was rejected for an entirely different reason. Lath cut from ingot iron looks exactly like lath cut from ordinary steel sheets. An architect or contractor purchasing lath for an important contract and paying an additional price for the best material, wants something more than the assurance of the manufacturer that he is getting the quality he is paying for. He wants the assurance of his own sense. The lack of this cannot be overcome with ingot iron.

Four types of galvanized lath were considered—hot galvanized, electro galvanized, Sherardized, and lath cut from a galvanized sheet. None of these require description except the Sherardized coating.

This process was invented in England by Mr. Sherard Cooper-Cole, a noted metallurgist, some seven years ago. The metal to be treated is packed in zinc dust and baked for several hours at a temperature just below the melting point of zinc. The process and the result is very similar to case hardening. Just as the casting while being case hardened absorbs part of the carbon in which it is packed, so the metal lath while being Sherardized absorbs zinc. The process differs from case hardening in the formation also of a pure zinc coating on the outside of the steel, while no corresponding coating of carbon is formed on a casting.

As galvanizing offers a perfect solution to the difficulty regarding scratches, the investigation now began to narrow down to a choice between these four types. Tests, however, which will be described later, were being made at this time which showed the probability of considerable damage to the zinc by plaster. It was therefore suggested that the coating should be lead instead of zinc, as the formation of lead oxide would protect the lead coating from further destruction. Mr. G. Frank Allen, a noted metallurgist, was consulted. He reported that the suggestion was of no value because lead is electro-negative as compared with iron, and the iron would therefore be rusted at the expense of the lead. He proposed an alloy of lead and zinc mixed in such proportion as to be neutral toward iron. This was rejected as again opening up the difficulty regarding scratches.

The simplest manner of making galvanized lath is to cut it from a galvanized sheet. With Herringbone lath the results are very satisfactory, although the coating is frequently cracked at the bends. Whether it would prove so in the diamond mesh type, where the bends in the metal are much more frequent, remains a question. The process was rejected on account of its commercial impracticability. To a technical man who understands how galvanizing protects, there is no difficulty apparent from the raw edges of the strands. It would, however, require considerable explanation to sell such lath to the average layman. Plato once said that a man's reputation is more injured by telling an improbable truth than a plausible lie, and the statement is as true to-day as in Plato's time. If nothing better had been found the process would have been adopted, but was finally rejected.

Hot galvanizing was rejected on account of the cost. The result is not better than that obtained by Sherardizing, while the cost is about eight times as great.

Electro-galvanizing was rejected on similar grounds. This process is really zinc electro-plating. In order to be efficient there must be sufficient zinc deposited to supply the wasting while the electric currents due to hardening of the plaster are in progress, and still leave an ample residue for further protection. Perfect color is obtained with a quantity of zinc entirely insufficient to provide perfect protection. It is also much cheaper to provide color than protection, and unfortunately mere color satisfies the manufacturer. For this reason many American architects in specifying galvanized lath are now stating that it must not be electro-galvanized. By continuing the process any desired quantity of zinc can be deposited, but if sufficient is provided to give a coating equal to the Sherardized coating, the cost would be many times as great.

The Sherardized coating which was finally adopted differs from other types of galvanizing by reason of the zinc iron alloy coming between the pure zinc and the steel. Immediately beneath the surface will be found a thin coating of alloy which is almost entirely of zinc. Going further, the percentage of zinc decreases, while the iron increases until pure iron is reached. There is therefore no contact between two metals of appreciable variation in electric potential. The coating can be considered as made up of a large number of layers, each varying slightly in electric potential from those immediately above and below it. While electric currents doubtless exist in these layers, they seem to be very weak and very minute and their effect is quite insignificant. A given quantity of zinc is far more efficient in the Sherardized coating than when applied in any other manner.

Prof. Burgess of the chair of metallurgy of the University of Wisconsin, reports that a given quantity of filings from the Sherardizing alloy takes fifteen times as long to dissolve in acid as a like quantity of zinc filings from a hot galvanizing bath.

Another reason for choosing Sherardizing is that the process cannot be scamped and is practically proof against mistakes in the shop. The zinc dust in which the lath is baked is a very poor conductor of heat. The process starts at the outside of the drums long before it starts at the centre, yet heat must be kept up until the centre is being Sherardized. Conversely the process is still going on at the centre for some time after the drums are removed from the oven. The real protective is the alloy which is deposited before the pure zinc. If, therefore, pure zinc appears at the centre of the sheet, one can be sure that the process is perfect.

In order to assist in the selection of a proper coating an attempt was made to devise an accelerated test applicable to the case in hand. The commonly used acid tests do not represent anything like working conditions. Patent plasters are not acids.

A number of specimens were coated with pure plaster of Paris and kept for two weeks in a bath of exhaust steam. It was supposed that they would thus receive in a short time the effect of as much moisture and warmth as would ordinarily act on them in a long period of years.

The result was a disappointment. The temperature was above the melting point of asphaltum, and the cold japan coating had consequently been absorbed into the plaster. Plain uncoated lath ingot iron and cold japanned lath all looked equally rusted. All types of galvanized lath showed rust spots, although the electro-galvanized was much the worst of the lot. The samples were submitted to Thomas Heyes & Son, consulting chemists, for further examination. They reported that the only reason the galvanized coatings looked better than the bare lath is that iron oxide is red, while zinc oxide is white. Actually there was more loss through oxidization of the zinc than of the iron.

By this time evidence obtained from other sources had shown the superiority of the Sherardized process. It remained, however, to see whether the plaster test would develop any unknown weakness. The chemists were instructed to conduct a series of test on cold japanned and Sherardized samples. These were coated with pure plaster of Paris and carefully watched for five months. From time to time bits of the plaster were chipped off. The plaster film remaining in contact with the lath would then be scraped off and carefully analyzed. From the first these samples showed minute quantities zinc oxide and metallic zinc dust. This is doubtless loose dust left on the coating from the manufacturing process. The first test showed 0.18 per cent. of such dust, but no later tests showed as much as this. No trace of zinc sulphate or iron oxide was formed in any sample, although they were watched for carefully.

The cold japanned sample showed iron oxide at the exposed ends, while the edges of the Sherardized sample were clear. Otherwise the results from the cold japanned sample were as good as the other.

The chemist stated that "forever" is too long a time to be considered in a careful scientific statement. His tests showed, however, no reason to suppose that Sherardized metal lath in a gypsum plaster would not last for a number of generations.

In comparing galvanizing, the quantity of protecting zinc must always be considered. Prof. Burgess found that chemically pure zinc from an electro-galvanized coating from a hot process bath, and that the Sherardized alloy was fifteen times as efficient as the hot galvanizing zinc. This was for equal quantities of metal. The excess zinc or hot galvanized material is wiped off wherever this is possible. Average tests show the commercial Sherardized coating to be two to three times as efficient as a wiped hot galvanized coating, but the latter requires much more zinc. Such a hot galvanized coating is also superior to the electro-galvanized coating as commercially applied, yet if the same amount of zinc were used in both processes the electro-galvanized coating would be much the superior.

Metal lath when hot galvanized cannot be wiped off, and a very heavy coating of zinc is the consequence. Our tests would indicate the probable superiority of Sherardizing over even this very heavy coat, although the cost is only about one-eighth as great. A conclusive test of this point was made by Mr. J. H. Burn-Murdock in England. He had one-half a chain hot galvanized without wiping and the other half Sherardized. Both chains were hung in sea water. When they were finally removed the hot galvanized chain had lost all of its coating and was so badly rusted that some of the links could be broken in the fingers. The Sherardized chains had turned blue. There were occasional yellow patches which were rubbed off without showing pits or roughness below, and all the links were evidently as strong as when originally immersed.

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CEMENT CONCRETE IN HIGHWAY CONSTRUCTION.

BY W. A. M'LEAN.

Concrete is not a new material, having been employed centuries ago by ancient civilizations, and it is somewhat notable that their use of concrete was largely in association with highway construction. So durable have been some of the old Roman roads that they are still in use in Southern Europe, and are believed by the more ignorant peasantry to be of supernatural origin.

The modern use of concrete in highway work includes many details. It is largely used for pavement foundations, sidewalks, bridges and culverts, bridge abutments and retaining walls, bridge floors, curbs and gutters, manholes, catch-basins, handrails, and other special features. Its use for much of this work is so well known, and has been so frequently written about, that to attempt a complete description would be to repeat largely what has been already well said. There are points, however, upon which progress is being made, or with regard to which some problem remains unsolved, and which are therefore deserving of comment.

Concrete has become the recognized foundation for asphalt, brick and other high-class pavements for streets of towns and cities, and for the foundation of such brick pavements for country roads, as those so largely used around Cleveland, Ohio, in Cuyahaga County. Its use as a paving material for the surface of the street or road has, however, been looked upon with considerable uncertainty, although in a number of cases some degree of courage has been shown in carrying on this class of work.

Windsor has been the principal place in Ontario to adopt this type of paving, and reports as to its merits have, to the present, been favorable. Experimental blocks have been laid in Brantford, Chatham, Toronto and other cities.

Recently the good roads movement in the State of Michigan, as carried out in Wayne County, surrounding the City of Detroit, has led to the adoption of concrete for their country road construction. These roads are characteristic of main highways radiating from a large city. The requirements of this traffic have led to the adoption of a type of roadway consisting of a central concrete pavement from 10 to 18 feet wide. On each side of this are gravel or macadam shoulders making a road of the desired width, usually from 24 feet to 30 feet from outside to outside of gutters. The first of these roads, two miles in length, was built in 1909, but was found so satisfactory that in 1910 about 15 additional miles were completed or are under construction. The cost has varied according to local conditions, but, including shoulders, drainage and culverts, has been approximately $1.35 a square yard of concrete.

Concrete pavements are laid in much the same way as concrete sidewalks. The sub-grade is excavated and rolled. This should be carefully done, so that the monolithic pavement will have a base of uniform strength. If the sub-soil is of sand or gravel the concrete may be laid directly on it, but if it is clay or other impervious soil, it should have a layer, three or four inches thick of gravel, cinders, broken stone or other strong and porous material. The sub-grade should have the same camber or crown as the finished roadway is to have. If a town street, curbs are laid first; but if a country road, curbs are not used, the surface being merged into the gravel or macadam shoulders.

Over the sub-grade the concrete base is spread, to a depth varying with the amount of traffic, usually four or six inches. The mixture should be proportioned for greatest density, but common practice employs a 1:3:7 mixture of cement, sand and broken stone; or a 1:8 mixture of cement and gravel. The base should be immediately covered with a wearing surface 1½ or 2 inches thick of strong mortar or concrete.

At intervals of about twenty or twenty-five feet there should be expansion joints across the roadway, about one-half inch wide, and at the curb similar longitudinal joints, to be filled with pitch or asphalt.

To reach good results the greatest care must be taken in particulars now well known in paving and general concrete construction. As with any form of road construction, the drainage should be ample. The surface coat should be immediately spread on the base before the latter has commenced to harden, so that union will be complete. A well rolled and uniform foundation will prevent much cracking. The mixing of the concrete should be thorough, as imperfect mixing is the cause of a vast amount of poor concrete. The surface should be protected from the sun in the usual way with straw, sand, burlap, and should be kept moist until it is fully hardened. As a rule a concrete road or pavement should not be opened for traffic in less than ten days or two weeks.

A perfect pavement meeting all desirable conditions of cost, durability and service rendered has not yet been found, and as requirements in many cases are of an opposite character, it is not probable that an ideal pavement for universal use will ever be invented. The more important qualities usually sought in a pavement or road material are that it should be:—

(1) Low in first cost.
(2) Easily and cheaply maintained.
(3) Smooth and hard, so as to offer least resistance to traction.
(4) Easily cleaned.
(5) Noiseless.
(6) Not dust producing, and not muddy.
(7) Non-absorbent and sanitary.
(8) Such as to give a good foothold for horses.
(9) Comfortable for those driving on it.
(10) Neither glaring nor hot.

The principal objections to concrete as a material for roads and pavements have been that the surface is too hard and glassy to give a proper foothold for horses; that it is rigid and therefore hard on the feet of horses; that it reflects heat and has an unpleasant glare; that expansion joints chip at the angles and under constant traffic deepen to holes; that cracks due to defective construction and to uneven foundations are difficult to repair, and, like expansion joints, chip at the corners.

On the other hand, it has been recognized that were these difficulties overcome, concrete has certain merits. It is low in first cost as compared with other high-class pavements, and is one of the cheapest materials yet available for a permanent pavement or roadway.

My attention has recently been drawn to the concrete pavements of Ann Arbor, Michigan, which have been treated with a surface painting of tar over which a thin layer of fine gravel is spread just sufficient to be saturated and held by the tar.

The pavements are constructed in the usual way by the making process. They consist of a bottom layer of concrete 5 inches thick, composed of one part Portland cement to eight parts of sand and gravel, and a top layer of Portland cement 2 inches thick, made in the proportion of one part of cement to two parts of sand. An expansion joint 1 inch wide is left at each curb, and an expansion joint about ½ inch wide each 20 feet transversely of the street. After the concrete has partially set and is dry, there is spread a thin coating of hot coal tar or distilled coal tar, and before this is dry it is covered with a uniform layer of sharp sand. The transverse joints are filled with a mixture of pitch or tar and sand, care being taken to fill the joints flush with the surface of the pavement.

During 1909 there were laid 1,883 square yards of this pavement, and in 1910, so satisfactory was their experience, that there was laid 18,223 square yards, while petitions have been received for over 70,000 square yards to be laid during the present year.

For applying the tar, a steel tank holding about 450 gallons is used, under which is a firebox for heating the tar, and to the rear is a 2 inch gas pipe used for sprinkling the tar. When tar has been sprinkled over the pavement a rotary sweeper is driven over it, which distributes the tar uniformly over the street. The sand, which is passed through a ¾ inch screen, is then spread by a wagon having a triangular box 5 ft. long on the rear. Near the bottom of this box is a 2½ inch curved shaft, belted to the hub of the wagon. As the shaft revolves the sand is put on a screen, and by the time it reaches the pavement is nicely spread.

In appearance these pavements resemble sheet asphalt, the tar and sand coating overcoming the glare and reflection of heat. The tar and sand fills the expansion joints and depressions, serving as a wearing surface. The expansion joints disappear from view and do not chip at the corners. The tar-sand coating deadens the noise of traffic and makes the pavement impervious to moisture. If cracks appear these are painted with tar and sanded so that they, too, disappear and they do not crumble under traffic. The treatment, in short, overcomes the more objectionable features of the concrete pavement.

The pavements as heretofore laid in Ontario have cost about $1.15 a square yard. The tar treatment in Ann Arbor has cost two cents a square yard, and the treatment has been found to last for two years, making the cost of tarring and sanding 1 cent per square yard per annum. If further experience proves the success of this treatment, it should prove a boon to the smaller towns and villages in paving their main business streets. It should be useful for residential streets in large cities and would be exceedingly effective for main roads adjacent to large cities.

On the New York State highways an experimental road was last year built near Rochester of concrete cubes, in a manner somewhat similar to the German "Kleinpflaster" and to the "Durax" roadway of England. The New York cubes are two inches square, of gravel or stone concrete, and are turned out by machine. The road is first constructed with a good stone foundation; or, if an old macadam road, is re-shaped and crowned. The cubes are hauled and dumped like ordinary road metal, but are spread uniformly by hand. They are laid in a sand cushion, sand is swept into the joints, and they are then well consolidated by rolling. The cost on the experimental section was about 50 cents a square yard, or equal to the cost of a bituminous binder. The appearance when finished was very satisfactory, and it is hoped that the method will be found adequate for the requirements of rapid motor traffic, including resistance to wear and freedom from dust.

Concrete pavements, in addition to the mixing process, have been laid by a patented grouting method. A body of broken stone is first laid, is rolled to place, and is then flushed with a thin cement grout and rolled until consolidation is complete. By another process, not patented, the cement, sand and stone have been mixed and spread dry. This is spread to grade, is rolled to place, then is sprinkled and rolled until consolidation is complete and setting has commenced. It is doubtful if either of these processes meet the requirements of thorough mixing essential in making good concrete.

Concrete has by no means displaced steel for bridge construction, but that each has its place is the consensus of opinion reached by bridge engineers. The field of concrete bridges, reinforced with steel, is largely confined to spans up to 50 feet, where either flat or arched design may be employed. Concrete arches are commonly applicable to situations up to 100 or 125 feet span, provided there is sufficient rise, but the use of arches is in all cases restricted to situations where the abutment foundations are strong and unyielding. For spans up to 50 feet, flat top concrete bridges with vertical reactions are suitable for situations where the arch could not be employed, but even here a stronger foundation is essential than would be required in the case of a steel truss bridge. Slight movements of the abutment are ruinous to a concrete arch, are permissable to a slight extent with flat or beam concrete bridges, though to a less extent than would work serious injury to a steel truss.

Temperature stresses have been one of the serious difficulties in the design of concrete bridges. For short spans up to 50 feet these may be disposed of by the use of steel well distributed through the concrete. The amount of steel required in an arch will, of course, depend upon the design of the arch, but in a general way one per cent. of steel is usually required to take care of temperature stresses, particularly in spandrel walls. For longer span bridges expansion joints become necessary in the spandrel walls over the haunches and at the crown. These are objectionable in several ways. They weaken the bridge, but it is chiefly as regards the appearance to which exception can be taken. To the user of the bridge the expansion joint has the appearance of a crack, and creates a feeling of distrust in the strength of the bridge. These expansion joints are usually filled with asphalt or pitch to keep out moisture, but this treatment does not conceal them, and some other means of dealing with this detail of bridge construction would be welcomed by engineers.

There is a tendency to improve the appearance of shorter concrete bridges by the use of ornamental hand-rails. This is desirable, but failure frequently occurs through neglect of temperature stresses. Open rails should be broken into lengths of eight or 10 feet, given expansion joints and the ends of each span supported on a post. Between the posts spindles of various types may be used. The writer has seen a bridge of over 100 feet length in which much attention was given to appearance, but the hand-rails were not given expansion joints and every spindle in the bridge had been sheared.

In the design of concrete arches there are architectural details too often overlooked. The parapet or coping should be slightly arched, otherwise it will look concave. The wing walls should not be straight, but should be, if possible, slightly curved, as otherwise they look weak.

Surface treatment of concrete is all-important. The surface should be a part of the concrete itself. A mere plastering of a defective surface will not permanently remedy the defect. In securing a good finish, the frame-work should be dressed and closely fitted. The concrete mixture should be wet; should contain sufficient sand, and should be carefully spaded against the form work so as to avoid air holes and cavities. A good finish can be obtained by using fine granite in the surface mixture, this to be treated with acid or brushed with wire before the concrete has fully set. This treatment is not practicable except for special work, but in all cases a smooth and close grained surface can be obtained by using sufficient care. The practice of making the exposed surface of stronger concrete is good, and is readily accomplished by carrying up a board a few inches inside the surface to retain the core concrete until it is partially set, when the board can be removed and the surface concrete put in place.

A common fault in short span country bridges and culverts is that they are too short. As a rule it will be found as cheap to carry tile entirely across the road, using shorter head walls and widening out the fill, than to use a shorter length of tile with necessarily higher and longer head walls to retain the earth fill. Short span bridges on country roads should always have a width of at least 20 feet to accommodate traffic. It is an improvement in the appearance of such short bridges to extend the wing walls almost straight, carrying them to the top of the roadway and putting on a suitable hand-rail. The public see little but the hand-rail of short bridges, and if it is strong and of a good appearance, are impressed thereby. A bridge need not be ornamented, but is often a chief feature of the landscape and should not disfigure it. Though not ornamented, it can be ornamental.

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CONCRETE SURFACE TREATMENT.

BY ROBERT CATHCART.

The cement age is in its primitive stage, but its future is destined to eclipse all modern idea or thought. In the natural sequence of things and conditions no other building material offers such advantages to the builders of our national homes and factories.

A primitive age usually has many things to discover and work out for the future knowledge of the coming generations, but as we live in the age of progress in art, literature and science, the world looks to us to make these primitive conditions ideals to theirs, in accordance with their thought and advancement.

Thus we have before us the problem of concrete surface treatment.

The plans and modes of surface exterior treatment to large, massive work, such as bridges, abutments, reservoirs, etc., sometimes to exterior factory construction, are operated by both the mechanical and chemical treatment.

Mechanical treatment is executed either by skilled or unskilled labor, or mechanical devices, in the following manner: picked, scraped, rubbed when green, bush hammered, tooled, sand blasted, etc. All of the treatments are more or less expensive; in many cases the workmen have stunned the corners, and the surface generally gathers dust and dirt from the atmospheric gases, which tend to destroy or disintegrate the surface in time. They generally are for effect only, and give no great weather resistance to the surface. In fact they are unfavorable to many conditions of construction and in some cases only a makeshift to betterment.

Chemical treatment is also an unskilled or skilled labor treatment, with commercial muriatic acid, acetic acid, etc., diluted in water, consisting of cutting the scum surface to relieve the hidden aggregates. It is also a more or less expensive operation, and the non-uniformity of the surface, combined with its chemical action, does not put it into special favor, although such processes have been patented. The muriatic acid is liable to stain the surface, and leave salts that will form a soluble alkali in combination with the concrete and produce an efflorescence on the surface.

The veneer treatment to outside walls, with plaster, has proven quite unsatisfactory for many reasons. The bond to the concrete has generally been weakened by the percolation of water through the plaster to the concrete surface which has caused a loss of adhesion, strength and permeability.

The outward influences, such as the sun's rays, frost, rapid wetting and drying, cause shrinkage of the veneer surface, loosening all soundness of adhesion, resulting in cracking and scaling.

Thus it would look as if no plaster should be applied to exterior walls where dampness could possibly lodge itself.

Coated surfaces are those, generally speaking, that have had cement grouts or washes applied to them. These are never stable and are generally used to fill up the unevenness of the surface without regard to their lasting qualities. In fact, they are dangerous surfaces to apply any surfacing materials to because of the unstableness of the bond.

Floor surfaces are tricky, because no man finishes the entire area of surface, and they should be rigidly covered by inspection before operating any treatment.

Obstructions or obstacles to concrete surfaces that demand treatment for preservation and service:

External influences on concrete surfaces, such as heat, cold, sunshine, water pressure, percolation, show their effects in two forms of disintegration—mechanical and chemical—defined as follows:

Mechanical: A consequent washing away of the softer constituents from the surface by the softening action of water, due to frost, the alternating action of the sun's rays, resulting in a periodical expansion of the other surface.

Chemical: The proportion of carbonic acid gas, oxygen, water and gaseous products of combustion in the atmosphere, a separation of scales, sometimes thin and thick, which by accumulation and an infiltration of strange matter into the pores cause a peeling from the surface. In other words, you get a condition of disintegration or corrosion, from both within and without, which has been termed "wall rot proper."

The lodging of the soluble salts upon the surface is efflorescence, and mainly due to such conditions as above mentioned, although local conditions and materials govern its area of trouble.

The Need of Coatings and Kind of Materials.

To produce a material for surface coating on concrete has been widely discussed pro and con by all scientific bodies. Plaster has been found to be unreliable for exterior surface; grouts are unstable.

To obtain an alkali and acid-proof vehicle with a pigment that will have sufficient wearing body, has been the aim of the engineer, contractor, consumer and manufacturer. Thus we will consider a few of the essential factors:—

1. Materials should contain a vehicle and pigment working in harmony with the conditions upon the surface.

2. They should contain an acid and alkali sun-proof color and pigment.

3. They should be sufficiently heavy to fill the surface voids and stop suction.

4. They should have a sufficiently hard wearing surface to allow successive coats, without further treatment of the surface.

5. They should produce a finish sufficiently close to the texture and originality of the surface.

TREATMENTS.

Treatments of concrete surfaces are divided into two divisions, as follows:—

1. The shallow or superficial method of treatment is a filling of the voids near the surface without discoloration. It is one that was much sought after in the early stages of concrete work, especially on concrete blocks, etc.

Although the monolithic type of construction has gained more favor in larger construction, the mechanical and chemical treatments of surfaces, as outlined, are the surfaces most in need of a shallow or superficial treatment.

Cement bricks, blocks, cast stone, are also surfaces that need a light treatment.

2. The physical or external method or treatment is a coating of natural materials as a preventive of contact between the elements and the construction. The treatment should be defined under two divisions:—

(1) Damp-proofing and decorative, without complete obliteration of the texture of the surface.

(2) Water-proofing only, without the decorative feature, and complete obliteration of the texture of the surface.

Under the first division you have a combination of results against the mechanical and chemical disintegration of the surface as defined for exterior surfaces. The operation should be twofold in its purpose. Damp-proofing and decorative in one operation, without destroying the texture of the surface.

Under the second division comes the subject of water-proofing (defined by Webster as proof against penetration), which may mean everything or anything, and we are sure the word has been handled improperly as to its meaning, because of our undying enthusiasm on the subject, although it may only mean damp-proofing, instead of water-proofing.

Water-proofing is an engineering problem of much discussion pro and con and does not enter into the subject of concrete surface treatment.

SPECIFICATIONS.

The following essentials should be strictly adhered to to insure results for the surfacing of concrete:—

All exterior or interior surfaces must be free from loose scales, sand, grit, grease, oil or other foreign matter. Surfaces can be freed of such materials by either scraping, wire-brushing or scrubbing with carborundum brick.

No muriatic acid or other acid treatment should be used, unless the surface is afterwards thoroughly neutralized by water or a light alkaline solution.

Green surfaces should be treated with a thin coat of surfacing materials to aid the neutralization of the free lime, and to fill the voids upon the surface.

After the first coat is applied, evaporated patches should be re-coated to insure evenness on the future coats.

All surfaces should be dry and free from moisture, to give perfect adhesion.

Exterior surfaces of buildings should be protected from rain or heat while drying.

COSTS.

Physical or external treatment: The figures, including the cleaning and preparing the surface under ordinary conditions on two-coat work, are generally estimated at 25c to 40c a square yard.

The covering of a concrete surface is about one-half that of lead and oil on wood, first coat; and two-thirds of lead and oil on wood, second coat; and equal to lead and oil on wood on the third coat for exterior surfaces.

The labor for the physical coat treatment is about one-third greater than the application of lead and oil on wood for exterior surfaces.

The non-uniformity of surfaces, filling of voids and suction, explains the covering coats.

On concrete floors the covering of a concrete coating or dressing runs from 200 square feet per gallon, first coat, to 400 square feet, second coat, according to condition of surface.

No one job is an example or guide—they all differ in uniformity of conditions.

The shallow or superficial method or treatment on coating two coats costs about one-third to one-half per square yard less than the superficial or external treatments, depending upon conditions.

The operator must be careful of the first or priming coats, as they are the foundations for a perfect bond and neutralization of the surface for successive coats.

The operator should produce a surface texture without destroying the originality of the surface.

WARNING.

No surface is free from dirt and foreign matter, and it requires the removal of such substances either by wire brushing or a light acid treatment, to remove stained portions in order to secure a firm bond and penetrating quality to the surface.

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INSPECTION OF CONCRETE.

BY E. A. JAMES.

Although concrete was a material of construction in the days when the Roman Empire flourished, yet we speak of it as one of the modern materials of construction. For several centuries other materials were more convenient and more fashionable, and with the settlement of America the plentiful supply of timber and the ease with which it was made suitable for construction work, marked timber the cheap material of construction, and it was not until the '70's that cement and concrete again entered into construction work in any great quantities.

To-day the annual consumption of cement in Canada is close unto five million barrels, which represents in completed structure many times five million dollars' worth.

Where the uses are so varied as are those to which concrete is placed, it is but natural to expect to hear of failure, and where the amount of money expended annually is so large it is reasonable to suppose that every effort should be made to insure good work.

In concrete work, as in other classes of construction, good design and good materials are necessary, and granting that we have good design and good materials, a class of well-skilled workmen should produce a finished material of a high order.

Concrete, however, has suffered more than any other building material because of inferior workmen, and this has been brought about largely by those interested in the early days in the sale of cement, who claimed as one of the chief reasons why it should be largely used, was the cheapness of placing, because it did not require skilled laborers.

Fortunately we do not hear so much of the claim as formerly, for it is well recognized that workmen can, with the same ingredients, produce various grades of concrete.

Although among the first of the modern concrete structures many were well and substantially made, yet there were a number of serious failures, and because the engineers themselves were not any too familiar with concrete construction, and because of this recurring failure and the fact that most of the workers in concrete were men imperfectly skilled, a system of careful inspection was inaugurated.

While the necessity of inspection depends to a certain extent upon the nature of the material and the production, it is always a question of just when and where, and to what extent it pays to inspect. In concrete work inspection must be made of the component parts. The sand, the gravel, the cement and the water must be examined, as well as the concrete while it is being mixed and poured.

The scope and possibilities of a system of inspection enlarges greatly with specialization.

Inspection should be planned to accomplish at least expense the best results, which may be numerated something as follows:—

1. To prevent loss or defects by accidents or delays.
2. To prevent loss of time and material on work already beyond repair.
3. To prevent the necessity of replacing defective work.
4. To prevent decrease in quality because of the demand for increase in quantity.
5. To point out imperfections in alignment, methods and material.
6. To record proper allowances for unavoidable extras.
7. To draw the attention of the superintendent to workmen who must be better instructed or trained.
8. To stimulate good will through fairness, in fixing responsibilities.

Inspection organized to cover any one or all of these purposes, will be similar in personality, varying only with the degree of perfection required.

Before it can be determined just when and where it pays to inspect, the following conditions must be satisfied:—

1. Responsibility must be fixed with certainty.
2. The inspection must not cause unnecessary friction.
3. The inspector must have to do with quality only, not design.
4. The responsibility for defective work must be placed upon the workman as well as upon the inspector.

Responsibility Must be Fixed with Certainty.

As inspection has for its purpose the pointing out of the defects, it is necessary for the inspector to be able to locate the cause of the defect. One of the most foolish things that can be done is attaching blame to the wrong person, and unless it is possible to discover immediately just when and where the cause of the defect lies, the fixing of responsibility is very difficult. It is therefore necessary to have the material on the ground in sufficient time for thorough inspection before it is mixed, for even after the inspector has detected the defect, the responsibility is not necessarily fixed. The error may be due to wrong specification, poor material, defective measurement, defective mixing, or even unsuitable weather conditions. It is therefore necessary that instructions and specifications be in writing.

Inspection Must Not Cause Unnecessary Friction.

No system of inspection which would simply complain of defects, without attempting to trace the cause or to assist in the improving of conditions, will be of any assistance. It certainly would not tend to happy relations between contractor and engineer. So it becomes necessary, if full benefits are to be derived from rigid inspection, not only to point out the defects, but the inspector should be in a position to trace the cause and to suggest a remedy. Defective work must be detected as soon as possible, so that the conditions under which the work was done may be fresh in the workman's mind, and the responsibility with certainty attached to him when the defect is through careless workmanship.

The Inspector Must Have to do with Quality Only, Not Design.

To point out defects will not necessarily stop the repetition, and although it may be the duty of the inspector to trace the cause, fix the responsibility and suggest the remedy, the inspector must not have to do with applying the remedy or of interfering with the workmen.

When the inspector has reported defects in material or workmanship to the engineer and contractor, or their representatives, he must content himself with awaiting the corrections through the proper officials, althought it should be within his power to stop or reject the work until there is an opportunity for investigation, and to take upon himself these responsibilities he must have knowledge equal to that of the superintendent of the work.

Responsibility for Defective Work Must be Placed Upon the Workman as Well as Upon the Inspector.

While we have stated that the inspector should not interfere with the workman, yet the knowledge that he is present will have a disciplinarian effect and will prevent the sacrifice of quality for quantity. Inspecting alone will not reduce bad workmanship to a minimum, but the workman must be supplied with proper tools, proper instructions, and must be trained in his work and held responsible for the quality of his work. He must be trained to inspect his own work. We have known cases where the men were paid a bonus for saving cement, and where this is the case it almost requires as many inspectors as men to secure compliance with specifications. Where it is known that the contractor is encouraging his men to skimp the work the inspector should lay his information before the engineer, and at once vigorous measures should be taken to remove such contractor from the work, for he will not do good work, no matter what the inspection is.

PLAN OF INSPECTION.

There are many plans of inspection, any of which may get good results, and all of which may fail in securing good results. Inspection depends more on the inspector than upon the method.

You may have inspection by central bureaus which retain men who are experts in their line of work who report first to their bureaus, or to the engineer in charge of the work, as may be arranged.

You may have inspection by your own local staff or foreman, or you may require such guarantees that inspection at the end of one or two-year period will be all that is necessary. The plan of inspection to be adopted will necessarily depend on the character of work being carried out.

THE DUTIES OF THE INSPECTOR.

It should be the duty of the inspector to see that all forms are erected on the lines laid down by the engineer, that these forms are stiff and well braced, and that all material and workmanship are in accordance with specifications. He should look after the removal of forms and see that the concrete is not injured in the removal.

Aside from concrete walks, form work is the most difficult to get properly placed, and it is much easier to develop a good inspector out of a good carpenter than out of a good concrete worker.

If the work is to be done at night under artificial light it will be necessary to increase the staff of inspectors, for concrete that can be detected in the day time, by color, will not show lack of proper mixing of materials under artificial light. In fact, where high-class work is required, or in finishing surfaces, as a rule it is better not done at night at all.

The cost of inspection is variable, being in some cases as low as one per cent. of the total cost and as high as two and one-half per cent. I think it is a usual thing two per cent. should be allowed for inspection, and good inspection is cheap at that price.

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A FEW POINTS ON REINFORCED CONCRETE DESIGN.

BY C. S. L. HERTZBERG.

In designing reinforced concrete structures one is continually meeting minor problems, upon which very little satisfactory information can be obtained from the numerous treatises on the subject. In the following paper I shall endeavor to enumerate a few points in design which are sometimes apparently not given the attention they deserve.

Footings have probably given more trouble to the designer, the erecting contractor and the owner, than anything else in connection with reinforced concrete. Unequal settlement in footings is responsible for numerous unsightly deformities and cracks and some collapses.

The common type of reinforced concrete column footings is, of course, easily dealt with, and differs from a plain concrete footing only in its being designed as a flat slab to resist bending instead of being sloped off as a pedestal. In this type of footing the centre of pressure coincides with the centre of gravity of the footing area, and the required size is formed directly from the load to be carried and the resisting power per square foot of the soil. Trouble is sometimes caused by having a footing too large in comparison with the size of the other footings in the same building. This is particularly liable to happen in the design of wall column footings in the following manner:—

If the footings are designed to carry the total dead and live load, figuring each floor of the building fully loaded, then the interior footings will, under probable loading, not stress the soil as highly as will the wall column footings. The reason for this is, of course, that the load figured to come on the wall column footings is usually about 70 per cent. dead load (which is present under all conditions) and 30 per cent. live load (which is never all there), while that figured on the interior column footings is generally about 40 per cent. dead and 60 per cent. live. As the live load on the footings of a building of five storeys or more is never more than 50 per cent. of the total live load, it will readily be seen that the pressure per square foot is less on interior footings than on exterior ones.

As all soil is compressed under any loading, the interior footings will not settle as much as the exterior ones, and the result is sometimes the cracking of floor beams and slabs.

The difficulty is overcome, to a certain extent, by the custom of reducing the live load by about 50 per cent. in buildings of over a certain number of storeys. This, however, would appear to be insufficient, and it would seem that either a greater reduction should be figured in designing interior footings, or else no reduction at all should be allowed in figuring exterior column footings. It would also appear to be wise to even add a small percentage on corner column footings, as a much larger portion of the wall coming on these is dead weight.

It very often occurs that the footings under wall columns cannot be built to extend beyond the outside line of the column. In cases of this kind some sort of combination footing should be used. This is sometimes done by carrying the column in question on a cantilever beam, pinned down at the other end by one of the other columns. In this type care must be taken to reduce the footing under the second column in proportion to the upward thrust from the end of the cantilever beam.

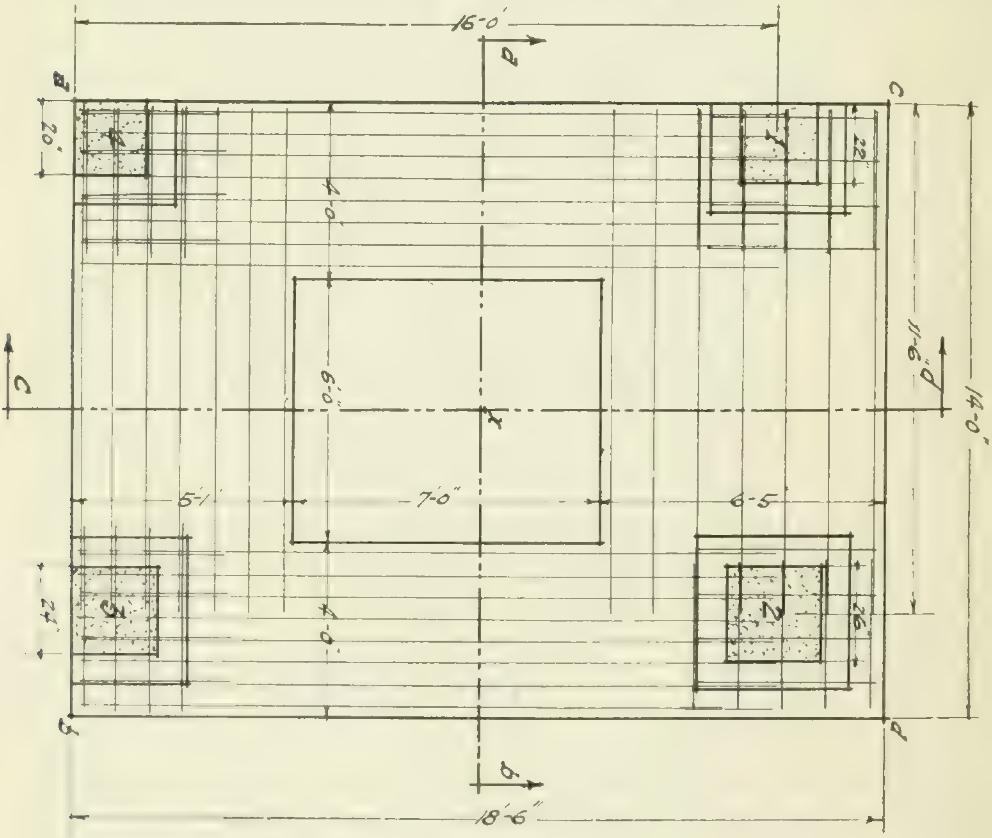

FIG. 1—Plan.

A simpler method of treating the above is as follows:—

Consider the wall column in question and the nearest interior column as acting together on a combined footing. Figure the loads coming on both columns and find the position of their resultant load. Add the two loads and divide by the soil value per square foot. This will give the required footing area. Design a footing of this area and varying in width from one end to the other in such a way that the centre of gravity of the area will coincide with the point of application of the resultant from the two column loads. The thickness of the footing and the reinforcing material must now be designed, treating the footing as an inverted beam, supported at the two columns and resisting the upward pressure of the soil, which will be of an intensity per square foot equal to the soil value, minus the weight per square foot of the concrete in the footing.

The above method can be used for designing combination footings for any number of columns.

Figure 1 shows a footing of this type designed to carry the four columns indicated, whose loads were (1) 267,000 lbs., (2) 347,000 lbs., (3) 284,000 lbs., and (4) 197,000 lbs. The soil value assumed was 5,000 pounds per square foot. Column 1 was a corner column, 2 and 3 were wall columns, and 4 was an interior column. The footings could not extend beyond the lines a b and a c. The footing was designed as follows:—

Sum of column loads = 1,095,000 lbs.

Sum of moments about side a b = 10,272,166 foot pounds.

Therefore centre of pressure is $\frac{10272166}{1095000}$ = 9 ft. 4½ in. from a b.

Taking moments about line a c, we find centre of pressure is 7 ft. from a c.

This locates the point x, the centre of pressure.

Area of footing required $\frac{109500}{5000}$ = 219 square ft.

The lengths 18 ft. 6 in. and 14 ft. of the sides a c and a b are now arbitrarily assumed.

Area of rectangle a b c d = 259 sq. ft.
Area of footing required = 219 sq. ft.

Area to be deducted = 40 sq. ft.
Deduct area e f g h 7x6 = 42 sq. ft.

Let x = distance from a c to centre of gravity of area to be deducted.

Let y = distance from a b.

Then x $= \frac{259 \times 7 - 217 \times 7}{42}$ = 7 ft.

And y $= \frac{259 \times 9.25 - 217 \times 9.38}{42}$ = 8 ft. 7 in.

which locates the position of the area of e f g h, which will give a footing whose centre of gravity coincides with the centre of pressure x.

The footing was then designed for bending by treating it as four beams between the four columns, figuring on 5,000 pounds per square foot upward pressure, minus the weight of the concrete in the footing.

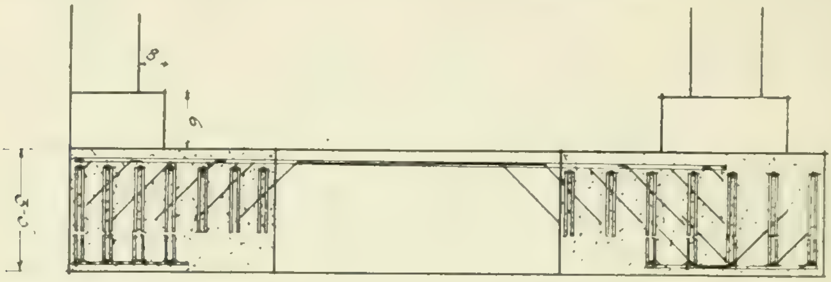

Fig. 2—Section on line A-B.

While the centre of pressure will, of course, shift under different conditions of column loading, still the variation cannot be sufficient to cause a serious settlement of any part of the footing.

In some cases it is very difficult to economically combine a wall column footing with any other footing. Where this is the case the footing is increased towards the inside of the building and along the wall. When this is done the column must, of course, be tied in at the top and figured to resist the bending caused by the eccentric loading on the footing. This bending is generally increased by the bending moment from the eccentricity brought on the column from the floor loads.

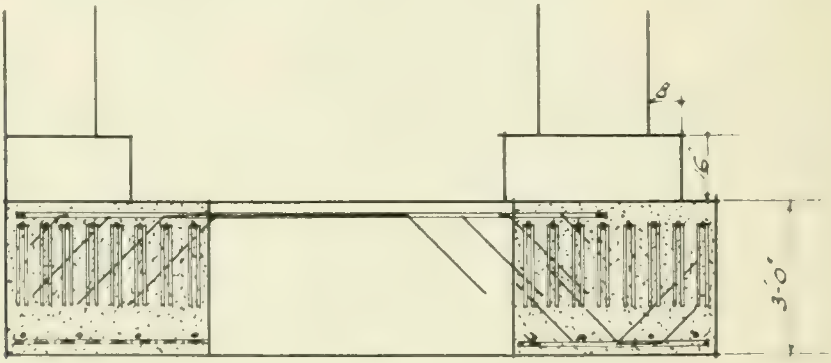

Fig. 3—Section on line C-D.

A point in designing reinforced concrete which is often overlooked is the bending produced in wall columns carrying long span beams. This moment seldom gives trouble in the lower tiers of columns in a building of any considerable height, as in such cases the columns are so heavily loaded that the eccentricity is not sufficient to produce actual tension in the outside of the column.

The common practice of designing wall columns 20 per cent. heavier than interior columns does not always overcome this tendency to crack from bending, as the extra strength is not applied in the proper place.

Consider the roof columns of a building of considerable width in which the roof beams span from side to side with no intermediate support. The usual custom is to carry the column reinforcement to within a few inches of the top of the roof slab, and to bend the anchor bars of the beams down into the columns the usual depth to prevent cracking in the upper surfaces of the beams near the ends. In a building designed in this way the result is pretty sure to be cracks across the outer surfaces of the columns immediately under the level of the bottoms of the beams, even though the roof be under no load other than the dead load of the structure itself. The reason for this is that the beam deflects under its own weight and the weight of slab carried. This deflection produces a tension in the upper surface of the beam at the end, which tension is also present at the outer surface of the column where it is altogether liable to produce large cracks. These cracks can be seen in many buildings. They should be provided against by increasing the reinforcing steel in the outer side of the column. This reinforcement should not be stopped at the roof level, but should be bent in along the upper surface of the beam. The use of plain steel rods for reinforcement in these columns (if the same are not bent over as described) increases the liability to crack, owing to the fact that they must be embedded to a greater distance than deformed bars in order to develop their tensile strength. Cracks of this nature are, of course, more unsightly than they are dangerous, for beams supported in this manner are usually designed as non-continuous over the supports and should be of the required strength whether pinned down to the columns or not. However, the bond with the column is an added strength to the beam and should be preserved.

The placing of brackets under a beam of the above description does not overcome the difficulty, and is, in my mind, poor practice. The brackets tend to spread the columns by causing the beam to act as an arch whose thrust is not properly taken care of, and cracks will very likely occur on the outer surface of the columns under the brackets. This construction acts, to a great extent, like a roof truss without a tie rod.

Reverse bending should be given particular attention in the design of highway bridges where heavy moving loads have to be provided for. In short span culverts where a flat slab is used this reverse bending at the abutments, if not properly taken care of, may result in a failure which has all the appearance of a shear failure, and such it may be after a certain point, although it has probably started in tension cracks in the upper surface of the slab.

Consider a culvert, say 12 feet clear span, to be designed to carry a 15-ton road roller. The slab is designed as non-continuous, and enough steel is inserted in the bottom to give a resisting moment to properly take care of the total maximum bending moment liable to come on the culvert. In all probability the concrete itself will figure to take care of all the shear at 50 pounds per square inch, and therefore no extra provision is made against failure through shear.

At first glance this culvert would appear to be properly designed to insure against failure from any cause, for, as the slab is not figured continuous over supports, it does not seem necessary to put any steel in the top of the slab at the abutments. This conclusion would be safe if the slab were cast separate from the abutments, but if (as is nearly always the case) the abutments and slab are monolithic the following is liable to occur:—

A heavy vibratory load comes to the centre of the span and produces considerable deflection, and as the slab is tied down to the abutment, tension is produced in the upper surface of the slab and on the outer surface of the abutments. The slab, being thinner than the abutment, cracks on top just inside the line of the abutment. Then as the load approaches this point the shear is increased and the cracked concrete is probably not capable of resisting this shear and collapses. This failure might have been prevented in three ways, namely, by the use of top steel, by the use of steel shear members, or by having a complete horizontal joint between the slab and abutment.

The advantages of what is known as flat ceiling construction are many, the most desirable among them being the appearance produced and the economy in floor height. The chief disadvantage in the most common types is our lack of scientific data on the subject. In a well-known type opinions differ nearly 100 per cent. as to the bending moment to be figured in slabs under the same loading. In the Engineering Record of 24th December, 1910, there is an account of some measurements made to obtain the strain existing in different portions of a flat slab floor under working loads. From these strains the existing stresses are figured. The results of these measurements appear to indicate that some designers are over-sanguine about the carrying capacities of this type of floor.

A more conservative design of flat ceilings is effected by increasing the width of the beams and decreasing their depth until the underside of the beam is flush with the underside of the slab. The slab in these cases is usually made up of small reinforced concrete joists, with tile fillers in between and two or three inches of concrete over the top to aid in compressive resistance.

In this type of floor the stresses are known and the strength can be figured along the same lines as the ordinary slab and beam construction. The tile fillers are placed as much as possible below the neutral axis of the slab, so as not to decrease the compressive resistance of the concrete, and of course they decrease the dead load of the floor. This type of floor is not as economical in steel as the usual slab and beam construction on account of the decreased arm of the resisting couple of the steel in tension and the concrete in compression.

Two-way reinforcement in a rectangular panel, designed according to the accepted theory of reductions in bending moments, effects economy in concrete only. If the bending moments each way be reduced in the usual manner of multiplying by $\frac{B^4}{A^4 + B^4}$ for the shorter span and by $\frac{A^4}{A^4 + B^4}$ for the longer where A represents the shorter span and B the longer, the steel may be slightly reduced by placing less near the edges of the panel than near the centre. This reduction is, however, offset by the fact that, in using bar reinforcement the amount of resistance of the upper layer of steel is decreased by the decrease in the resisting arm of the forces. The saving in concrete is, of course, effected by figuring it to take its full working compression in two directions at right angles to one another.

Before closing I would like to enter a plea for the standardization of unit stresses and formulæ in reinforced concrete design throughout Canada. Some things, of course, cannot be standardized, but such points as ratio of the modulii of elasticity of steel and concrete, the allowable working compressive strength of concrete, both in bending and in direct compression, the limits of tee action, etc., might be definitely settled and adhered to by all designers. If it is safe in one city to design a continuous beam uniformly loaded to resist a bending moment of one-twelfth WL, then it is equally safe to do the same in the next city, despite the fact that the second city insists on it being designed for one-eighth WL. Other points might also be straightened out, such as whether a specification should insist on using 12 for the ratio of the modulus of elasticity of steel to that of concrete, when in another part it calls for a working stress of 350 pounds per square inch for the concrete in a column and 10,000 pounds for compressive steel embedded in the same column.

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ADAPTATION OF CONCRETE TO LONG SPAN BRIDGES.

BY FRANK BARBER.

For the longer concrete arches, especially those in which the span exceeds 150 ft., a type has been developed which differs considerably from the more common arch of short span. But before proceeding to point out some of the characteristics of long span arches, it might be interesting to notice how the concrete arch of the present day compares in length with the steel arch and truss bridge.

There are about fifteen steel arch bridges of spans over 500 feet, the longest of these being the Upper Niagara, span 840 feet, and the Viaur, France, 720 feet, in span. Also about fifteen simple truss bridges have been built over 500 feet in span, of which the longest is the Municipal Bridge, St. Louis, span 668 feet.

Many lists of concrete arches have been published, the list of 500 concrete and stone bridges of all lengths, by Prof. Malvert A. Howe, in "Symmetrical Masonry Arches" being specially worthy of mention. None of these, however, that are known to me is up-to-date. Accompanying this is a list of concrete arches, plain and reinforced, over 175 feet span, which I believe is correct to date. It is remarkable that all of them have been completed within seven years.

I have notes of thirty-four masonry arches all over 150 feet span, fifteen of which exceed 200 feet; and some nineteen concrete arches over 150 span.

It will be understood that in the present discussion by the length of the bridge is meant its clear span or the span of its longest arch. There are bridges very much longer than those in the list just read, which, however, are supported by many piers resting on the bed of the stream. One of these, joining the mainland of Florida with one of the Florida Keys, now nearing completion, is about 2½ miles of continuous concrete bridge. Mere length in a bridge when it is made up of short spans is not a claim for merit, and though it may be a distinguishing mark which carries interest, it forms no part of our present discussion.

The first great masonry span was built in the fourteenth century, when some bold engineer flung across the River Adda in Italy a span of 251 feet. This was soon after destroyed, but the span was not exceeded for over 500 years. The beginning of the more recent activity in long arch construction was in

CONCRETE ARCH BRIDGES OVER 175 FT. SPAN

YEAR	NAME OR LOCATION	ENGINEER	SPAN	RISE	FLOOR WIDTH	TYPE	REFERENCE
1910	Grafton St., Aukland, New Zealand	B. F. Moore	320′	....	36′	*a*, *b*	N. vol. 64, p. 113 E. Jan. 17, 1910
1910	Rocky River, Cleveland	A. M. Felgate	280′	80′	60′ 4″	....	R. vol. 61, p. 4
1908	Sitter, Switzerland	Prof. E. Morsch	259′	87′	22′ 6″	*a*	R. vol. 62, p. 133 B. Feb. 13, 1909
1908	Walnut Lane, Philadelp'a	Harry H. Quimby	233′	70′ 3″	60′	....	R. vol. 56, p. 222 C. vol. 32, p. 202 N. vol. 57, p. 117 T. Dec., '09, p. 423
1904	Grunwald, Germany	Prof. E. Morsch	229′ 7″	42′	30′ 2″	*a*, *b*	N. vol. 153, p. 199
1905	Ulm, Germany	....	215′	18′ 7″	46′	*b*	N. vol. 55, p. 307
1907	Kempten, Bavaria	....	211′ 7″	86′ 8″	54′	*b*	N. vol. 57, p. 307
1910	Meadow St., Pittsburg	....	209′	46′ 2″	50′	....	R. vol. 55, p. 575
1908	Fogaras-Kronstadt R'lw'y, Hungary	Prof. Konstauntin Zielinsky	C 197′	70′	....	*a*	N. vol. 62, p. 22 B. May 29, 1909
......	Almendares, Cuba	....	190′	....	....	....	....
1907	Lautrach, Bavaria	....	187′ 6″	32′ 3″	13′ 9″	*b*	N. vol. 57, p. 480 R. vol. 55, p. 575
1904	Decize, France	....	184′	15′ 4″	35′	*b*	A. 4, Tri., 1905
1907	Pyramont, France	M. S. DeMollins	C 177′	25′	12′ 6″	....	G. Jan. 16, 1909 N. vol. 59, p. 353
	UNDER CONSTRUCTION						
	Rome, over R. Tiber	....	328′	32′ 9″	....	....	....
	Monroe St., Spokane, Washington	J. C. Ralston	281′	115′	68′	....	N. vol. 62, p. 242
	PROJECTED						
	Henry Hudson, New York	Moisseiff	703′	177′	80′	*a*	N. vol. 58, p. 559 R. Nov. 16, 1907
	Larrimer Ave., Pittsburg	....	300′	....	....	....	
	Medland, N.Y., for barge canal	Wm. R. Davis	285′	53′ 5″	129′	....	N. vol. 63, p. 274 R. vol. 61, p. 265

(*a*) Arch reinforced with steel. (*b*) Three-hinged. (*c*) Centre to centre. REFERENCES (N) Engineering News; (R) Engineering Record; (C) Engineering-Contracting; (A) Annales des Ponts et Chausses; (B) Schweizerische Bauzeitung; (T) Trans. Am. Soc. C. E.; (G) Genie Civil; (E) Engineer, London.

1833, when two arches of 200 feet were built, one the Grosvenor Arch at Chester in England, and the other at Turin in Italy. These held the record for length until 1864, when the Cabin John Arch of 220 feet was completed at Washington, D.C. This span was not exceeded until 1902, when the Luxemburg Bridge in Germany was built, of span 277 feet. The palm then passed to the present holder for the longest stone arch, the Plauen Bridge, Saxony, 295 feet, finished in 1905 by the Engineer Leibold.

It is only within the last decade that concrete has challenged stone for record spans. The longest arch existing is of concrete, the Grafton Bridge, New Zealand, span 320 feet, and this span will soon be exceeded by one over the Tiber at Rome of 328 feet. The two longest and finest concrete arches in America are the Walnut Lane, Philadelphia, 233 feet span, and the Rocky River, Cleveland, of 280 feet. The Sitter Bridge, in Switzerland, designed by Prof. Morsch, is in the class of those just mentioned, all of which have been completed since 1908.

The longest concrete arch so far projected is the great Henry Hudson Memorial Arch of span 703 feet, which may, however, never be built. Only two steel arches exceed this in span. For longer spans than these recourse has been had to cantilever or suspension bridges.

We have not yet built any very long span concrete bridges in Canada. I shall mention the first four to show our present developments. The longest is the Wadsworth Arch, Weston, span 118 ft. 6 in., designed by our firm, Barber & Young. Next to this is the span of 116 ft. over Kettle Creek, St. Thomas, designed by Jas. Bell. The third in length was built last year at Coaticook, Que., span 100 feet, and after this is the arch at Massey of nearly the same span, designed by W. A. McLean, of the Public Works Department.

It will be noticed that the long arches are generally designed to have open spandrels, whereas the common short arches have solid masonry or concrete spandrel walls, and the roadway is supported by solid earth filling resting upon the arch ring. The reason for the open spandrels is partly that the calculations for design cannot be exact with earth filling over the arch ring. The exact amount of horizontal thrust on the arch ring due to earth filling is not known, and moreover, the weight of earth varies with the season and the amount of moisture contained. But earth filling is prohibited in long arches of concrete chiefly on account of its great weight, for this would make a long arch unnecessarily massive and expensive.

Another point of design which may be mentioned in this connection is the possibility of using hollow abutments. In this case the main wall and wing walls of the abutments are in the shape of the letter "U." In the ordinary abutment wing walls are designed to resist the earth pressure from the inside, but in the hollow abutment design the wing walls have merely to support the floor system and are consequently very much thinner and cheaper. There are at least two cases where hollow abutments have been used.

A further and later development of the type we are considering is the use of several arch ribs instead of a continuous ring. One advantage of the ribbed arch which can be immediately seen is the possibility of simpler construction on the upper part of the bridge. In this design, one row of columns resting upon each rib is used to support the floor system, whereas with the continuous ring many more columns, or even continuous spandrel walls, are necessary in order to distribute the weight of the floor system as evenly as possible over the arch ring. Another feature of the ribbed arch is that the floor is cantilevered over the outside ribs, so that the out-to-out width of the arch ribs is always considerably less than the total width of floor. The virtue of this lies in the narrow abutment required.

In order to discuss another point regarding the ribbed arch, we must first get a clear idea of the structural function of the arch. An arch rib or ring is first of all a column under compression throughout its length from skewback to skewback. Further than this, it is a beam, and must resist the bending moment induced by shifting live loads and by temperature changes. Now we know that a beam of rectangular section will resist bending better when set on its edge than when it is laid flat on its side between its supports. For the same reason a concrete arch when made into two or more deep ribs will resist bending due to live loads much better than if the same amount of concrete should be spread out in a comparatively thin continuous ring.

On the other hand, temperature stresses can be resisted better by a thin arch ring than by the comparatively thick ribs, but time does not permit me to enter into the mathematical reasons for this. Thus for a given quantity of concrete, the deeper and narrower the rib, the better for resisting live load moments, but the worse for resisting temperature stresses. Between these two opposing tendencies, for any given set of conditions as regards span, rise and loading, it is possible with the ribbed arch to work out the economical section for the rib, and this should always be computed for each case. But with a continuous ring its width is fixed by the width of roadway, and the most economical section is not possible. This must not be considered as typical of present practice, for I know of but one case, that of the Wadsworth Arch, where the economical section has been computed, but we may look for a future advance in this line.

A feature of some long arches, which is adopted more especially by European designers, is the use of pivots or hinges in arch ribs. Ribs may have one, two or three hinges. If a single hinge is used, it is placed at the crown, and if two hinges, at the skewbacks. But the best design for hinged arch ribs is to use three hinges, one at the crown and one at or near each skewback. These hinges may be cast steel or iron, working upon pins, or they may consist simply of thin sheets of lead, which, on account of their malleability, will permit all the motion that is expected to take place. With three hinges temperature stresses are entirely eliminated, but the stresses resulting from live loads cannot be so well resisted as in the arch with no hinges. A discussion as to the relative merits of hinged and hingeless arches would be too mathematical for our present purpose. American practice, even for the longest arches, is to use no hinges.

I sometimes hear an opinion thoughtlessly expressed by some lay brother, that the longer the arch, the more need of reinforcement. This is contrary to the truth. In order to get a clear idea of the reason for this we shall consider the effect of three sets of forces on the arch ring; those due, first, to live loads; secondly, to changes of temperature; and lastly, to the dead weight of the bridge itself.

Let us consider for a moment the nature of these different kinds of stress. When a live load is placed over the haunch of an arch the latter acts as a beam, so that the intrados of the arch under the load is in tension, and the whole arch at that point deflects downward. But downward deflection at this point must be accompanied by an upward deflection of the arch on the other side of the centre, and consequently the extrados of that part of the arch will be in tension. Secondly, a rise of temperature has an effect on the arch exactly similar to squeezing the two skewbacks together and slightly bending the whole ring. This causes tension of the extrados at the crown and tension of the intrados at the skewbacks. A fall of temperature has the opposite effect, and those parts of the arch which were before in tension will then be in compression. Thirdly, as to dead load stresses, when the arch is economically designed the dead load line of pressures will lie along the neutral axis, which means that the whole arch is in compression and that this compression at any radial section is equal on all parts of that section. In order to express this in less technical language, let us disregard for a moment the effect of live loads and temperature changes, and let us suppose the arch ring made up of a number of voussoirs or building blocks separated by a number of imaginary radial joints. When the curvature of the arch is correct all these blocks will bind together under the weight of the bridge, so that the whole rib is in compression throughout its length.

Thus we see that live loads and temperature changes have a tendency to cause tension, but when the bridge is properly designed the dead load can cause nothing but compression. It will be noticed that the tensile stresses due to live load and temperature changes are of a temporary nature, but the dead load compression is constant. As these three influences may all act together, the tensile stresses may at times in certain parts of the arch exceed the dead load compression. The net result of this will be tension in some part or parts of the ring. Wherever this occurs or is likely to occur steel reinforcement must be used, for as we know, concrete cannot of itself economically resist tension.

We have just seen the nature of the stresses caused by the three sets of forces, whether tensile or compressive, temporary, or permanent. We shall now consider for a moment their magnitude, and how the magnitude of each set of forces increases or remains stationary as the span of the arch increases.

In a small arch the stresses due to a live load, consisting of, say, a twenty-ton roller, may be greater than those resulting from the weight of the bridge itself, but as the span becomes greater the effect of this roller becomes comparatively slight compared to the greater dead weight of the bridge. For instance, a twenty-ton roller could have only a trifling effect on the great Walnut Lane Arch (except on the floor system, which we are not now considering). Thus as the span increases, the effect of a live load does not increase in the same proportion.

As to the temperature changes, we may lay down this principle. Temperature stresses vary inversely as the rise of the arch, and directly as the thickness of the rib, other things being equal. This is not the exact law, but it is near enough for our present purpose. Thus in flat arches temperature stresses will be greater than in high arches, other things being equal. Now, as the span of an arch increases in general, so will the rise become greater and also the depth of the rib. It is quite possible for the span, rise and rib thickness to be all increased in such a way as to leave the temperature stresses unchanged. It is difficult to lay down a general law, though we are safe in drawing this conclusion, that as the span of an arch increases temperature stresses do not increase in the same proportion.

On the other hand, the weight of the floor system, other things being equal, will increase in about the same proportion as the span increases, and the weight of the arch ribs in a still higher ratio, this rate being more nearly according to the square of the span. Thus as the span of an arch increases, the weight of the whole bridge and consequently the dead load stresses, increase in an accelerating ratio.

From the previous discussion it will be seen that as the arch increases in length the dead load compression also increases still more, but the tensile stresses due to live loads and temperature do not increase in proportion to the length. Thus as the span increases a certain span will be reached where the tensile stresses can never exceed the constant dead load compression. For arches of this and all greater spans no tension can exist in any part of the rib, and if steel is used at all it will not be to resist tension, but to aid the concrete in resisting compression and to bind the whole together. As an example of this we may take the 259 ft. Sitter Arch in Switzerland, in which, according to the calculations of Prof. Morsch, no tensile stresses ever occur. This is one of the most important features which distinguishes long from short arches.

In the longer arches we have seen that the principal stresses are due to the weight of the concrete itself, and if engineers are to keep on designing arches of ever-increasing spans, it will become necessary to use a concrete capable of resisting high unit stresses. In this connection it is interesting to notice the experiments of the President of this Association, Prof. Peter Gillespie, with concretes rich in cement. He has shown that the amount of the large aggregates, so long as the voids are filled, has little effect on the strength of the concrete, which seems to depend altogether on the strength of the mortar. Further, the strength of the mortar seems to vary approximately with the amount of cement, a mixture of one of cement to two of sand being stronger than when one of cement is used to three of sand, and so forth, until the sand is entirely eliminated, when the strongest mortar is the result consisting of neat cement. The strongest concrete is therefore made with large aggregate and cement enough to fill the voids without any sand. Gillespie's concrete, by which we may denote the sandless variety, on account of its greatly increased strength, was found under certain circumstances to be cheaper than leaner concrete. It would be well worth while to conduct experiments to discover a method of so grading course aggregate as to produce a minimum of voids. In this case it would appear that the void could be filled with less cement and the strength of the concrete not be impaired. I think that Gillespie's concrete will be largely used in days to come for very long span arches.

The method of concreting the arch ring is very different in long span arches from that used on shorter ones. The concrete is first placed in large voussoirs with small spaces between them, and after this concrete has set the small key-spaces are concreted, completing the arch ring. By this method, before the small key-spaces are filled in the shrinkage from setting has taken place, and since the main weight of the ribs has been taken by the false work, all the settlement that is possible will have occurred. Thus no danger or cracking is to be expected from either shrinkage of concrete or slight settlement of forms.

The architects have made us painfully aware of the fact that a style of architecture suitable to concrete buildings has not yet been evolved. This in my opinion does not apply to the type which has been developed for long arches. Let us take the Walnut Lane Bridge as an example of this type. The design is not only beautiful and simple, but it is inevitable and results from the distinctive characteristics of concrete. This masterpiece in concrete presages a time when only the things shall be considered best whose makers have dedicated their work to Time.

Canadian Cement and Concrete Association

STANDARD BUILDING REGULATIONS FOR REINFORCED CONCRETE

Adopted by the Association, March 9th, 1911

I. GENERAL.

Height of buildings.

1. There shall be no fixed limit to the height of buildings of reinforced concrete except that arising from the requirements of these regulations.

Plans and computations.

2. Before permission is granted by the Building Department to erect any reinforced concrete building, complete plans, accompanied by specifications, signed by the Engineer and Architect, must be filed with the Building Department and remain on file for public inspection until the building is completed.

3. The Building Department shall have access to the computations. These shall give the various loads assumed, such as dead, live and wind loads, the impact allowance, if any, and the resulting maximum stresses.

Specifications.

4. The specifications shall state the qualities of the materials to be used for making the concrete, and the density and strength required.

Certificate of carrying capacity.

5. Upon the completion of the building the Engineer and Architect shall issue, under the approval of the Building Department, signed certificates, to be posted on each floor of the building, stating the safe carrying capacity per square foot.

Records.

6. There shall be kept, for the inspection of the Building Department, careful records of the progress of the work. These records shall show the date of placing of all the concrete, the date of removal of the forms, and they must be turned over to the Building Department when the building is completed.

Walls.

7. Reinforced concrete walls may be used in place of brick or stone exterior or bearing walls with

thickness reduced to two-thirds the thickness specified for brick walls. Such walls shall not be less than four (4) inches thick.

8. Reinforced concrete walls shall be reinforced at both faces. The maximum spacing of bars in the same vertical plane shall be 18 inches on centres. The quantity of metal employed shall be sufficient to provide for a uniformly distributed horizontal pressure of 30 pounds per square foot acting on either side of the wall.

II. Materials.

Cement.

9. Only Portland Cement shall be used in reinforced concrete structures which meets the requirements of the specifications of the Canadian Society of Civil Engineers.

10. Tests of the cement used in building operations shall be made from time to time under the supervision of the Building Department and in accordance with the preceding specifications.

Fine aggregates.

11. Fine aggregates shall consist of sand, crushed stone or gravel screenings graded from fine to coarse and passing, when dry, a screen of quarter-inch mesh. Not more than six per cent. shall pass a screen having 100 meshes to the lineal inch. They shall be of clean, silicious material free from vegetable loam or other deleterious matter.

Mortar.

12. Mortars composed of one part Portland Cement and three parts fine aggregate when made into briquettes shall show a tensile strength at least equal to the strength of 1:3 mortar of the same consistency and age made with the same cement and standard Ottawa sand.

Coarse aggregates.

13. Coarse aggregates shall consist of inert material graded in size, such as crushed stone or gravel, which is retained on a screen of quarter-inch mesh. The particles shall be clean, hard, durable and free from all deleterious material. The maximum size of the coarse aggregates shall be such that they will not separate from the mortar in laying and will flow readily around the reinforcement and into all parts of the forms.

Cinder concrete.

14. Cinder concrete shall not be used for reinforced concrete structures; it may be used for fireproofing. Where cinders are used as the coarse aggregate, they shall be composed of hard, clean,

vitreous clinker, free from sulphides, unburned coal or ashes.

Medium steel.

15. Medium steel for reinforcement shall have an ultimate tensile strength of 58,000 to 66,000 pounds per square inch, an elastic limit of not less than one-half the ultimate strength and a minimum percentage elongation in 8 inches of 1,500,000 divided by the ultimate strength. In all respects not covered by this specification, medium steel shall conform to the requirements of the General Specifications for Steel Bridges and Viaducts of the Department of Railways and Canals, Canada, 1908.

High elastic limit steel.

16. High elastic limit steel for reinforcement shall be made entirely from new billets and shall have an ultimate tensile strength of 80,000 to 96,000 pounds per square inch, an elastic limit of at least half the ultimate strength, and a minimum elongation in eight inches of 1,200,000 divided by the ultimate strength. It shall bend cold 180 degrees around a pin the diameter of which is four times the thickness of the bar without showing fracture on the outside of the bend.

Rust, scale and dirt.

17. All loose rust, scale or dirt shall be removed from reinforcement steel before it is placed in the work.

III. Design.

1. *General Assumptions.*

18. The ratio between the moduli of elasticity of steel and concrete shall, for a concrete crushing at 2,000 pounds per square inch in 28 days, be assumed as 15.

19. The compressive stresses in a reinforced concrete beam shall be assumed to vary directly as the distance from the neutral axis.

20. The tensile resistance of concrete shall be assumed to be zero.

T-Beams.

21. Where adequate bond between slab and web of beam is provided, the slab may be considered as an integral part of the beam, and its effective width may be determined by the following rules:

(a) It shall not exceed one-fourth of the span length of the beam.

(b) Its overhanging width on either side of the web shall not exceed four times the thickness of the slab, nor twice the thickness of the web.

(c) Where the slab extends on only one side of the beam, it shall be permissible to consider as effective flange area, an overhanging width not greater than four times the thickness of the slab or twice the thickness of the web.

2. *Bending Moments.*

Moments on slabs and beams.

22. In computing the positive and negative moment in beams and slabs continuous over several supports, due to uniformly distributed loads, the following rules are recommended:

(a) That for floor slabs the bending moments at centre and at support be taken at $\frac{w\,l^2}{12}$ for both dead and live loads, where w represents the load per lineal unit and l the span length.

(b) That for beams the bending moment at centre and at support for interior spans be taken at $\frac{w\,l^2}{12}$ and for end spans be taken at $\frac{w\,l^2}{10}$ for centre and adjoining support, for both dead and live loads.

Distribution of loads on slabs.

23. Floor slabs shall be designed and reinforced as continuous over the supports. The distribution of loads longitudinally and transversely in slabs reinforced in both directions shall be determined by the following formula:

$$r = L^4 : (L^4 + B^4)$$

where r is the ratio of the load carried transversely to the whole load, L is the length and B the breadth of the slab. If the length of the slab exceeds 1.5 times its width, the entire load shall be assumed to be carried by the transverse reinforcement. Square slabs are preferably reinforced in both directions using reinforcement in each direction calculated for one-half the specified load.

Distribution of loads on beams.

24. The loads carried to beams by slabs which are reinforced in two directions will not be uniformly distributed to the supporting beams, but may be assumed to vary as the ordinates on the sides of an isosceles triangle, the base of which is the length of the beam. The moments in the beams should be calculated accordingly.

3. *Working Stresses.*

25. The working stresses proposed below are for a concrete the ultimate crushing strength of which at an age of 28 days is 2,000 pounds per square inch. For concrete of greater or less compressive strength, proportionate increments to, or reductions from, the proposed stresses shall be made.

Bending.

(a) The maximum allowable compressive stress on the extreme fibre of beams shall be 650 pounds per square inch.

Direct compression.

(b) The maximum allowable compressive stress on concrete in columns without hoops, or in columns reinforced with less than one per cent. of hooping, shall be 500 pounds per square inch.

Pure shear.

(c) Pure shearing stresses, uncombined with tension or compression shall not exceed 120 pounds per square inch.

Diagonal tension.

(d) Shearing stress combined with diagonal tension shall not be greater than 40 pounds per square inch.

Bond.

(e) The maximum allowable bond stress between concrete and plain steel shall be 60 pounds per square inch, and between concrete and deformed bars approved by the architect or engineer, 120 pounds per square inch.

Tension in steel.

(f) The tensile stress on steel reinforcement shall not exceed one-half of the elastic limit of the steel nor in any case 16,000 pounds per square inch.

Columns.

(g) Columns reinforced with longitudinal reinforcement only, shall be figured for a working stress in the concrete of 500 pounds per square inch and in the steel, a stress of 7,500 pounds per square inch.

(h) Columns with at least one per cent. of reinforcement in the form of bands or hoops shall be figured at a working stress of 500 pounds per square inch on the concrete.

(i) Columns reinforced with at least one per cent. and not more than four per cent. of longitudinal bars and with at least one per cent. of reinforcement in the form of bands or hoops, or with structural steel column units which thoroughly encase the concrete core, shall be figured at working stresses of 650 pounds per square inch on the concrete and 9,750 pounds per square inch on the longitudinal reinforcement.

4. *Details of Design.*

Spacing of bars.

26. The lateral spacing of bars should not be less than three diameters, centre to centre, nor should the distance from the side of the beam to the centre of the nearest bar be less than two diameters. The clear spacing between two layers of bars should be not less than one inch. In no case shall the minimum thickness of fireproofing be less than that specified by the Building Department.

Web reinforcement.

27. In the calculation of web reinforcement to provide a strength of 120 pounds per square inch, which is assumed to be the maximum permissible vertical shearing stress on beams no matter how well reinforced for diagonal tension, the concrete may be counted upon as carrying one-third of the shear, or for 2,000 pound—concrete, 40 pounds per square inch. The remainder may be provided for by means of metal reinforcement consisting of bent bars or stirrups, but preferably both. The area of a stirrup, acting in tension, must be proportioned to the force equal to the whole horizontal shear produced between the two adjacent stirrups less the amount to be carried by the concrete. This total horizontal shear may be found by multiplying the distance between the stirrups by the average horizontal unit shear, which is equal to the vertical unit shear, and is represented by the expression $\frac{V}{d}$, where V is the total vertical shear not carried by the concrete and d is the depth of the beam to the centre of the steel. In accordance with this method, the stress in a vertical stirrup will be equal to $\frac{Vs}{d}$ and in a stirrup or rod bent at 45 degrees it will be $0.7\frac{Vs}{d}$, in which s is the horizontal spacing of stirrups or bent rods in the direction of the axis of the beam.

Stirrups.

28. The longitudinal spacing of stirrups shall be such that at least one rod shall intersect below the centre of the beam any 45° line of rupture in

the portion of the beam where the shearing stresses exceed the allowable shearing value of the concrete.

Length of columns.

29. The ratio of unsupported length of columns to their least width shall be limited to fifteen.

Area of hooped columns.

30. The effective area of the hooped column shall be taken to be the area within the protective covering which shall not be less than two inches in thickness. In the case of columns reinforced with structural shapes, it shall be taken as the area wholly within the structural shapes.

Longitudinal reinforcement.

31. Bars composing longitudinal reinforcement must be straight and shall have sufficient lateral support to hold them securely in place until the concrete is placed.

Hooping.

32. The clear spacing of bands or hoops shall not be greater than one-eighth the diameter of the enclosed column. Adequate means must be provided to hold bands or hoops in place so as to form a column, the core of which shall be straight and well centred.

Bending stresses.

33. Bending stresses due to eccentric loads must be provided for by increasing the section until the maximum stress does not exceed the values specified above.

Bases of columns.

34. At the bases of columns, longitudinal reinforcement shall abut squarely on bearing plates, or shall be carried into the footing a sufficient distance to develop the compressive stress in the reinforcement on the basis of the working stresses specified above.

Splicing of reinforcement.

35. In columns, all longitudinal reinforcing rods shall be properly butted or spliced.

36. Wherever it is necessary to splice reinforcement by means of lapping, the length of the lap shall be determined upon the basis of the safe bond stress and the stress in the bar at the point of splice; or, a connection shall be made between the bars of sufficient strength to carry the stress. Splices at the point of maximum stress must be avoided.

Shrinkage and temperature changes.

37. Where large areas of concrete are exposed to atmospheric conditions, the changes of form due to shrinkage and variation of temperature must be provided for by placing small rods or light fabric near the exposed surface, of a quantity to

be determined on the basis of the elastic and thermal properties of the materials.

IV. CONSTRUCTION.

Measuring and mixing of ingredients.

38. The ingredients of concrete shall be thoroughly mixed to the desired consistency, and the mixing shall continue until the cement is uniformly distributed and the mass is homogeneous and uniform in colour.

39. Methods of measuring and proportioning the ingredients shall be such as are approved by the Engineer or Architect.

40. When the conditions will permit, a machine mixer of a type which insures the uniform distribution of the materials throughout the mass shall be used.

41. When it is necessary to mix by hand, the mixing shall be on a water-tight platform, and special precautions must be taken to turn the materials until they are homogeneous in appearance and colour.

Consistency.

42. The materials must be mixed wet enough to produce a concrete of such consistency as will flow readily into the forms and about the reinforcement, and which, on the other hand, can be conveyed from the mixer to the forms without separation of the coarse aggregate from the mortar.

Retempering.

43. Retempering mortar or concrete, i.e., remixing with water after it has partially set shall not be permitted.

Placing of concrete.

44. Concrete shall be rapidly and continuously handled from the time the water is added until it is finally placed in the work. The placing shall be facilitated by the use of suitable agitating and ramming tools, so as to produce a thoroughly compact concrete of maximum density.

Wetting.

45. Concrete exposed to premature drying shall be kept damp for a period of at least four days.

Forms.

46. Before concrete is placed, care shall be taken to see that the forms are substantial and thoroughly wetted and that the space is entirely free from debris.

Joining up new work to old.

47. When work is resumed, concrete previously placed shall be roughened, thoroughly cleaned of foreign material and laitance, drenched and slushed with a mortar consisting of one part Port-

land Cement and not more than two parts fine aggregate.

Placing in water.

48. Where concrete must be placed under water, unusual care must be taken to prevent the cement from being washed away.

Freezing temperature.

49. Concrete shall not be mixed or deposited at a temperature lower than 28 degrees Fahr. unless special precautions are taken to avoid the use of materials containing frost or covered with ice crystals, and to provide means to prevent the concrete from freezing after being placed in position.

Reinforcement.

50. The reinforcement shall be accurately located in the forms as required by the plans and carefully secured by wiring or other means against displacement before placing the concrete.

Joints.

51. Reinforced concrete work shall be stopped at points where joints will have the least possible effect on the strength of the structure.

(a) Footings shall, where practicable, be cast to their full depth at one operation.

(b) Columns shall be cast at a single operation and work shall be stopped at the underside of the lowest beams or girders bearing on the columns. Where a bracket is used, the bracket shall be considered a part of the beam or girder.

(c) Construction joints in beams and girders shall be vertical and at the centre of the span. Any concrete which may run past the bulkheads must be cleaned up before the concreting of the next section is started.

(d) Construction joints in slabs shall be at the centre of the span. Where a joint occurs between a slab and a beam or girder, no T-action in figuring the beam or girder shall be considered.

Removal of forms.

52. No forms shall be removed without the written consent of the Engineer or Architect.

Tile and joist construction.

53. Wherever floors are built with a combination of tile or other fillers between joints, the following rules regarding the dimensions and methods of calculation shall be observed.

(a) The ratio of minimum depth to clear span of joist shall not be less than one to twenty-four.

(b) Wherever a portion of the slab above the tile joist shall be considered as acting as a Tee-beam section, the slab portions must be cast monolithic with the joist and must have a minimum thickness of 2 inches. In such a case all regulations applying to Tee-beams shall apply to tile and joist construction.

(c) In no case shall the tile or other fillers be considered as bearing stress.

(d) Wherever porous tiles, or other materials which by their nature will absorb water from the concrete, are used between the joists, care must be taken to thoroughly saturate the tiles, or other materials, with water immediately before the concrete is placed.

PETER GILLESPIE,
J. A. JAMIESON,
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Canadian Cement and Concrete Association

Standard Building Regulations for Reinforced Concrete

Adopted March, 1911

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WM. SNAITH
Secretary-Treasurer
57 ADELAIDE STREET, EAST
TORONTO

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III. Design.

1. *General Assumptions.*

18. The ratio between the moduli of elasticity of steel and concrete shall, for a concrete crushing at 2,000 pounds per square inch in 28 days, be assumed as 15.

19. The compressive stresses in a reinforced concrete beam shall be assumed to vary directly as the distance from the neutral axis.

20. The tensile resistance of concrete shall be assumed to be zero.

T-Beams.

21. Where adequate bond between slab and web of beam is provided, the slab may be considered as an integral part of the beam, and its effective width may be determined by the following rules:

(a) It shall not exceed one-fourth of the span length of the beam.

(b) Its overhanging width on either side of the web shall not exceed four times the thickness of the slab, nor twice the thickness of the web.

(c) Where the slab extends on only one side of the beam, it shall be permissible to consider as effective flange area, an overhanging width not greater than four times the thickness of the slab or twice the thickness of the web.

2. *Bending Moments.*

Moments on slabs and beams.

22. In computing the positive and negative moment in beams and slabs continuous over several supports, due to uniformly distributed loads, the following rules are recommended:

(a) That for floor slabs the bending moments at centre and at support be taken at $\frac{w l^2}{12}$ for both dead and live loads, where w represents the load per lineal unit and l the span length.

(b) That for beams the bending moment at centre and at support for interior spans be taken at $\frac{w l^2}{12}$ and for end spans be taken at $\frac{w l^2}{10}$ for centre and adjoining support, for both dead and live loads.

Distribution of loads on slabs.

23. Floor slabs shall be designed and reinforced as continuous over the supports. The distribution of loads longitudinally and transversely in slabs reinforced in both directions shall be determined by the following formula:

$$r = L^4 \div (L^4 + B^4)$$

where r is the ratio of the load carried transversely to the whole load, L is the length and B the breadth of the slab. If the length of the slab exceeds 1.5 times its width, the entire load shall be assumed to be carried by the transverse reinforcement. Square slabs are preferably reinforced in both directions using reinforcement in each direction calculated for one-half the specified load.

Distribution of loads on beams.

24. The loads carried to beams by slabs which are reinforced in two directions will not be uniformly distributed to the supporting beams, but may be assumed to vary as the ordinates on the sides of an isosceles triangle, the base of which is the length of the beam. The moments in the beams should be calculated accordingly.

3. *Working Stresses.*

25. The working stresses proposed below are for a concrete the ultimate crushing strength of which at an age of 28 days is 2,000 pounds per square inch. For concrete of greater or less compressive strength, proportionate increments to, or reductions from, the proposed stresses shall be made.

Bending. (a) The maximum allowable compressive stress on the extreme fibre of beams shall be 650 pounds per square inch.

Direct compression. (b) The maximum allowable compressive stress on concrete in columns without hoops, or in columns reinforced with less than one per cent. of hooping, shall be 500 pounds per square inch.

Pure shear. (c) Pure shearing stresses, uncombined with tension or compression shall not exceed 120 pounds per square inch.

Diagonal tension. (d) Shearing stress combined with diagonal tension shall not be greater than 40 pounds per square inch.

Bond. (e) The maximum allowable bond stress between concrete and plain steel shall be 60 pounds per square inch, and between concrete and deformed bars approved by the architect or engineer, 120 pounds per square inch.

Tension in steel. (f) The tensile stress on steel reinforcement shall not exceed one-half of the elastic limit of the steel nor in any case 16,000 pounds per square inch.

Columns. (g) Columns reinforced with longitudinal reinforcement only, shall be figured for a working stress in the concrete of 500 pounds per square inch and in the steel, a stress of 7,500 pounds per square inch.

(h) Columns with at least one per cent. of reinforcement in the form of bands or hoops shall be figured at a working stress of 500 pounds per square inch on the concrete.

(i) Columns reinforced with at least one per cent. and not more than four per cent. of longitudinal bars and with at least one per cent. of reinforcement in the form of bands or hoops, or with structural steel column units which thoroughly encase the concrete core, shall be figured at working stresses of 650 pounds per square inch on the concrete and 9,750 pounds per square inch on the longitudinal reinforcement.

4. *Details of Design.*

Spacing of bars.

26. The lateral spacing of bars should not be less than three diameters, centre to centre, nor should the distance from the side of the beam to the centre of the nearest bar be less than two diameters. The clear spacing between two layers of bars should be not less than one inch. In no case shall the minimum thickness of fireproofing be less than that specified by the Building Department.

Web reinforcement.

27. In the calculation of web reinforcement to provide a strength of 120 pounds per square inch, which is assumed to be the maximum permissible vertical shearing stress on beams no matter how well reinforced for diagonal tension, the concrete may be counted upon as carrying one-third of the shear, or for 2.000 pound—concrete, 40 pounds per square inch. The remainder may be provided for by means of metal reinforcement consisting of bent bars or stirrups, but preferably both. The area of a stirrup, acting in tension, must be proportioned to the force equal to the whole horizontal shear produced between the two adjacent stirrups less the amount to be carried by the concrete. This total horizontal shear may be found by multiplying the distance between the stirrups by the average horizontal unit shear, which is equal to the vertical unit shear, and is represented by the expression $\frac{V}{d}$, where V is the total vertical shear not carried by the concrete and d is the depth of the beam to the centre of the steel. In accordance with this method, the stress in a vertical stirrup will be equal to $\frac{Vs}{d}$ and in a stirrup or rod bent at 45 degrees it will be $0.7 \frac{Vs}{d}$, in which s is the horizontal spacing of stirrups or bent rods in the direction of the axis of the beam.

Stirrups.

28. The longitudinal spacing of stirrups shall be such that at least one rod shall intersect below the centre of the beam any 45° line of rupture in

the portion of the beam where the shearing stresses exceed the allowable shearing value of the concrete.

Length of columns.

29. The ratio of unsupported length of columns to their least width shall be limited to fifteen.

Area of hooped columns.

30. The effective area of the hooped column shall be taken to be the area within the protective covering which shall not be less than two inches in thickness. In the case of columns reinforced with structural shapes, it shall be taken as the area wholly within the structural shapes.

Longitudinal reinforcement.

31. Bars composing longitudinal reinforcement must be straight and shall have sufficient lateral support to hold them securely in place until the concrete is placed.

Hooping.

32. The clear spacing of bands or hoops shall not be greater than one-eighth the diameter of the enclosed column. Adequate means must be provided to hold bands or hoops in place so as to form a column, the core of which shall be straight and well centred.

Bending stresses.

33. Bending stresses due to eccentric loads must be provided for by increasing the section until the maximum stress does not exceed the values specified above.

Bases of columns.

34. At the bases of columns, longitudinal reinforcement shall abut squarely on bearing plates, or shall be carried into the footing a sufficient distance to develop the compressive stress in the reinforcement on the basis of the working stresses specified above.

Splicing of reinforcement.

35. In columns, all longitudinal reinforcing rods shall be properly butted or spliced.

36. Wherever it is necessary to splice reinforcement by means of lapping, the length of the lap shall be determined upon the basis of the safe bond stress and the stress in the bar at the point of splice; or, a connection shall be made between the bars of sufficient strength to carry the stress. Splices at the point of maximum stress must be avoided.

Shrinkage and temperature changes.

37. Where large areas of concrete are exposed to atmospheric conditions, the changes of form due to shrinkage and variation of temperature must be provided for by placing small rods or light fabric near the exposed surface, of a quantity to

be determined on the basis of the elastic and thermal properties of the materials.

IV. CONSTRUCTION.

Measuring and mixing of ingredients.

38. The ingredients of concrete shall be thoroughly mixed to the desired consistency, and the mixing shall continue until the cement is uniformly distributed and the mass is homogeneous and uniform in colour.

39. Methods of measuring and proportioning the ingredients shall be such as are approved by the Engineer or Architect.

40. When the conditions will permit, a machine mixer of a type which insures the uniform distribution of the materials throughout the mass shall be used.

41. When it is necessary to mix by hand, the mixing shall be on a water-tight platform, and special precautions must be taken to turn the materials until they are homogeneous in appearance and colour.

Consistency.

42. The materials must be mixed wet enough to produce a concrete of such consistency as will flow readily into the forms and about the reinforcement, and which, on the other hand, can be conveyed from the mixer to the forms without separation of the coarse aggregate from the mortar.

Retempering.

43. Retempering mortar or concrete, i.e., remixing with water after it has partially set shall not be permitted.

Placing of concrete.

44. Concrete shall be rapidly and continuously handled from the time the water is added until it is finally placed in the work. The placing shall be facilitated by the use of suitable agitating and ramming tools, so as to produce a thoroughly compact concrete of maximum density.

Wetting.

45. Concrete exposed to premature drying shall be kept damp for a period of at least four days.

Forms.

46. Before concrete is placed, care shall be taken to see that the forms are substantial and thoroughly wetted and that the space is entirely free from debris.

Joining up new work to old.

47. When work is resumed, concrete previously placed shall be roughened, thoroughly cleaned of foreign material and laitance, drenched and slushed with a mortar consisting of one part Port-

land Cement and not more than two parts fine aggregate.

Placing in water.

48. Where concrete must be placed under water, unusual care must be taken to prevent the cement from being washed away.

Freezing temperature.

49. Concrete shall not be mixed or deposited at a temperature lower than 28 degrees Fahr. unless special precautions are taken to avoid the use of materials containing frost or covered with ice crystals, and to provide means to prevent the concrete from freezing after being placed in position.

Reinforcement.

50. The reinforcement shall be accurately located in the forms as required by the plans and carefully secured by wiring or other means against displacement before placing the concrete.

Joints.

51. Reinforced concrete work shall be stopped at points where joints will have the least possible effect on the strength of the structure.

(a) Footings shall, where practicable, be cast to their full depth at one operation.

(b) Columns shall be cast at a single operation and work shall be stopped at the underside of the lowest beams or girders bearing on the columns. Where a bracket is used, the bracket shall be considered a part of the beam or girder.

(c) Construction joints in beams and girders shall be vertical and at the centre of the span. Any concrete which may run past the bulkheads must be cleaned up before the concreting of the next section is started.

(d) Construction joints in slabs shall be at the centre of the span. Where a joint occurs between a slab and a beam or girder, no T-action in figuring the beam or girder shall be considered.

Removal of forms.

52. No forms shall be removed without the written consent of the Engineer or Architect.

Tile and joist construction.

53. Wherever floors are built with a combination of tile or other fillers between joints, the following rules regarding the dimensions and methods of calculation shall be observed.

(a) The ratio of minimum depth to clear span of joist shall not be less than one to twenty-four.

(b) Wherever a portion of the slab above the tile joist shall be considered as acting as a Tee-beam section, the slab portions must be cast monolithic with the joist and must have a minimum thickness of 2 inches. In such a case all regulations applying to Tee-beams shall apply to tile and joist construction.

(c) In no case shall the tile or other fillers be considered as bearing stress.

(d) Wherever porous tiles, or other materials which by their nature will absorb water from the concrete, are used between the joists, care must be taken to thoroughly saturate the tiles, or other materials, with water immediately before the concrete is placed.

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